HIGH AUTHORITY
OF THE EUROPEAN COAL AND STEEL
COMMUNITY

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The High Authority
of the
European Coal and Steel Community

Steel Congress 1964

Progress in Steel Construction Work

Luxembourg
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1 *Official Opening of the Congress*

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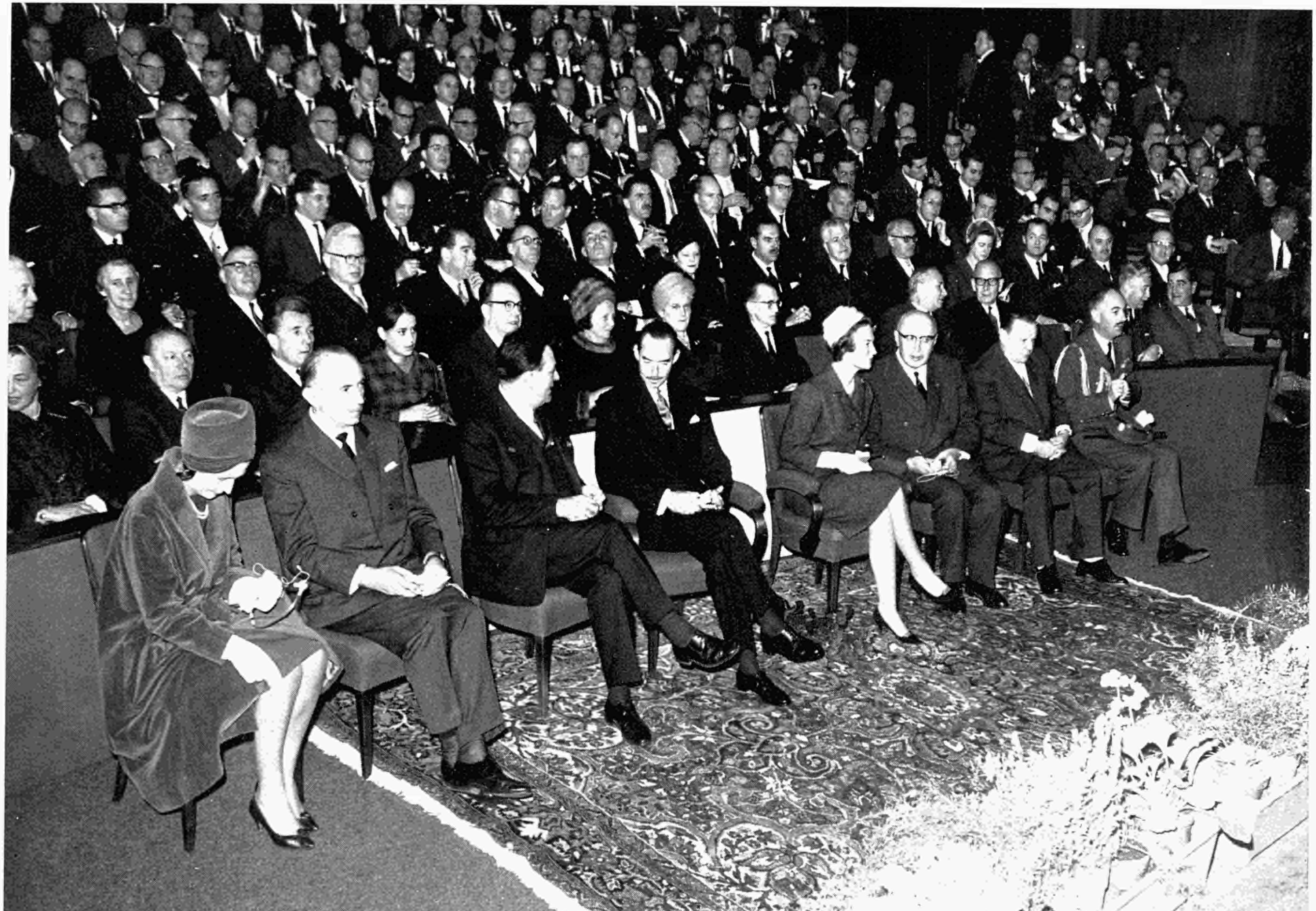
Their Royal Hignesses The Grand Duke and Grand Duchess of Luxembourg at the official opening of the Congress.



Dino DEL BO

President of the High Authority

I am here today simply to say a few words about the reasons which caused the High Authority of E.C.S.C. to decide to convene this Congress. In this connection I must naturally refer to the provisions of the Treaty of Paris, by which the High Authority is required to deploy its energies in the coal and steel sector, and so, within its terms of reference, help ensure in the national economies of the six Community countries the maximum improvement in the standard of living and the creation of more and ever more new jobs — in short, to help ensure prosperity.

Since 1952, the High Authority has so applied itself to its duties that very real progress has been made as compared with the position in which the six countries then were, when steel production was insufficient to match the requirements of reconstruction and of social advance. In other words, the great object was to enable enough steel to be produced to meet demand.

From 1952 to 1961, the combined labours of the Community workers, economic operators, technicians, responsible Civil Servants and others resulted in some exceedingly striking achievements. Crude-steel production rose from a mere 42 million tons in 1952 to 72 million in 1961, and production of special steels from 3 100 000 tons to 6 100 000. Also during this period, in accordance with the Treaty's requirements, the High Authority established rules on price publication, rules on publication of forecasts, rules on cartels and concentrations, while in addition it made available very substantial funds to aid investment and to finance essential technical research.

From 1952 to 1961 the member countries' efforts may be said to have been concentrated on enabling supply to balance demand. In 1961, however, a certain alteration in the trend began to develop. Steel production rose steadily in the six countries; so also did exports. But it was found that the flow of imports from third countries was starting to speed up.

The High Authority and the member States have always pursued a large and liberal commercial policy in regard to steel production : the E.C.S.C. tariff wall has always been, and still is, one of the lowest in the world. Nevertheless, it is necessary to safeguard the immense sums sunk in steel production, to safeguard also the livelihood of our workers, above all to remember that a basic producer industry such as steel is a vital component in European civilization and power, and to see to it that the Six shall not be a mere pawn in the game of opposing ideologies, but shall retain its ability to choose and act for itself.

The steel production of the third countries has been growing larger and larger. Now many of these countries enjoy a very considerable degree of tariff protection against imports. In some of them technology is highly advanced; others have the advantage of lower production costs, others again of lower-cost and higher-grade iron ores than the Community's.

Our problem now is once again to establish a balance between demand and supply. The High Authority, in accordance with the policy of the six E.C.S.C. Governments, declines to regard the protectionist solution as either appropriate or, more particularly, worthy of a politically responsible democratic Europe. On the contrary, the High Authority and the six Governments are convinced that what is needed is to create new forms of demand and develop new uses for steel, and that meeting these demands and using steel for these purposes will at the same time help to better the lot of men and women in all the countries of the world.

Such is the prime reason for this Congress. Our aim has been on this occasion — the first, we hope, of many — to focus the attention of scientists, technicians, specialists, Government representatives and public servants on one particular use for steel, its use in the building sector, in which new aesthetic values and new technical properties may well ensure a tremendous market for steel. We cannot foresee today what the results of the Congress will be, but the presence of those who have accepted the High Authority's invitation to attend, the presence of such men as Mr. Jeanneney, so outstandingly qualified not only in the field of economics but in the political interpretation of the problems raised by economics, makes the success of our labours here a foregone conclusion.

May I end with one observation. Iron and steel were, alas, very much to the fore at a time, only a very few years back, of which ineradicable traces still remain. Iron and steel as it were stood as the symbol of stubborn enmity among men and the instrument of their violence. We who are met here today, at this Congress for the study of new and peaceful uses for steel, wish that its peaceful use shall serve at the same time to bring about better understanding among peoples.

With this expression of our profound conviction and with your permission, I now declare open this first Congress on the Utilization of Steel in Construction Work.

Pierre WERNER

Prime Minister of the Grand Duchy of Luxembourg

On behalf of the Grand Ducal Government, I have pleasure in extending a most cordial welcome to all those attending the Congress on Steel Utilization.

We are honoured and gratified that you should have chosen our capital city for this important gathering, and we would express our appreciation to the conveners, the High Authority of the European Coal and Steel Community. In return we offer you, as we do to all who are good enough to visit us, our honest Luxembourg hospitality, born of friendly understanding, genuine international-mindedness, and belief in Europe. To you of the Congress we offer something more — an example and a performance in your own particular field, a country the stuff of whose economic life is iron and steel. One of our economists has gone so far as to write that "Luxembourg is a product of iron (*un don du fer*) as Egypt is a product of the Nile." Its production of something like thirteen tons per head of population beats all others by several lengths — one of the few quantitative records it is given to a small country to hold. Even amid the wide range of past and present applications of iron and steel, we feel we can point to notable examples, from the ornamental ironwork of the eighteenth century to the bold new structure of the Pont Grande-Duchesse Charlotte now in process of completion within a stone's throw of this Theatre, not forgetting the distinguished achievements of our fabricating shops and the marvels performed with reinforced concrete.

I venture to hope, then, that the atmosphere of this country and city may be favourable to your debates and aspirations.

There is another obvious reason for you to feel at home here. Since 1952 Luxembourg has been the seat of the first of the European Communities, responsible specifically for the Common Market in coal and steel. Here in this thousand-year-old city we have seen — and we Luxemburgers have grown accustomed to seeing — common internal and external policy hammered out, over the last dozen years, for six European countries which in 1964 are producing among them more than eighty million tons of steel. The sector-by-sector approach to European integration is no longer so much in favour as when Mr. Robert Schuman launched his historic appeal. Nevertheless, the fact remains that this first great cast of the European net came off — and came off because it brought within the Community compass the forges of war and peace.

Much discussion is being devoted at this time to the projected merger of the Community Executives, and ultimately of the Communities themselves. Now a concentration of powers in the hands of a single Executive may be a step in European development, but this does not obscure certain practical economic facts. A number of basic sectors in the fields both of goods and of services will occupy a specially prominent place in the deliberations of the future European Executives, just as they do in the concerns of national policies. They will need to be approached in a special manner. They include steel, the archetypal basic industry, and it is for this reason that my Government are anxious that the steel market should operate in accordance with its particular requirements, and with the spirit and letter of the rules laid down in the Treaty of Paris. Those rules have worked well. The European steel market has developed very satisfactorily during these past twelve years, especially as regards interpenetration among the Six, business stability and expansion of capital investment. I am sure you will understand that we should be alert to this aspect of European development, seeing that the steel industry was already in 1952 the mainstay of our established economy, now committed to what was reckoned at that time the "European experiment."

Steel is still supreme, despite stiff competition. Neither plastics nor the light alloys have yet managed to supplant it. World production is increasing at what some may deem a headlong, an excessive pace. While not pronouncing judgment on that pace, you note it; you draw the conclusions and seek to devise new openings. You are right to study possible new uses for steel — if other materials can encroach on its traditional fields, why should not steel, adaptable as it is to so many purposes, invade the preserves of others? For instance, I was struck to see the reference in your programme to the construction of temporary steel roads.

I wish your discussions every success. I may add that I feel it would be an excellent thing if regular exchanges could be organized on the subject.

The city of Luxembourg is proud to be for a few days the hub of the steel world, the forum for informed discussion on that metal which, to take up the wording of your programme head, represents the construction of the future. Luxembourg is happy to follow this lead and join in the work of building.

Fritz HELLWIG

Member of the High Authority

For the first time for some years, the production of the E.C.S.C. steel industry, will in 1964 show a renewed substantial increase. For four years, from 1960 to 1963, Community crude-steel production remained stationary at around 73,000,000 tons a year: this year it will total approximately 82,000,000.

However, this most heartening development must not obscure the fact that certain all-important things, on which our steel industry's future hinges, have altered during these years. Direct exports of steel by the Community play no part in the recent improvement: they are lower than in 1960. The share of Community steel exports in total world trade in steel amounted during the last few years to barely one-third. For the first time, indirect exports in manufactured form have exceeded net direct exports. The current increase in production is very much more due to the movement of internal demand.

As is clear from these few indications, the steel industry has for some years been definitely in a state of technical and economic flux. Technological advance, competition and shifts in the pattern of the world market have caused a process of rapid evolution, which is going on even in those sectors of production where as recently as ten years ago technical and economic development appeared to have settled in some sort into a rut.

This surprising trend is due to a variety of causes. Within the Community, we have the vigorous revival which set in after the war, and the good effects of the six countries action in joining to form a wider market for steel. The abolition of Customs duties and impediments to trade among these countries stimulated competition, and by allowing natural locational relations to take effect, served to reorient supply and demand.

Other, outside influences are also at work, which have by now affected in varying degree all the steel-producing countries. International competition has stiffened; this, the most powerful driving-force in technological progress, has resulted in major innovations at practically all stages of steel production — improved ore preparation, continuous casting, oxygen steelmaking, fully continuous rolling, to name only a few.

Changes in the raw-material situation have also faced the Community with new problems to master. New deposits have been opened up from which high-grade foreign ores are being furnished at lower cost than ever; the general drop in maritime freight-rates is favouring the transport of ore and overseas coal to our shores. Consequently, the Community steel industry derives less advantage from the presence of indegenous ore and coal. Numbers of large firms have reacted by building new plants in coastal areas.

This is not to say that the coast is the only possible future location for the industry. On the contrary, the changes in the world steel market themselves indicate the present and future importance of the Community manufacturing industries as a determining element in the location of steel production.

As regards the alterations in the sales situation, these began gradually and so almost imperceptibly: not until a year or two ago did the structural change in progress become clearly apparent.

To take the export market first: the turnround which has taken place there is due primarily to three circumstances:

(a) the emergence of new exporter countries,

(b) the development of a world surplus, and

(c) changes in relative competitive positions.

As a result of the installation of new capacity, world steel production potential has increased from 256,000,000 tons in 1953 to 480,000,000 in 1964. It is notable that the largest increases have been in Japan and in various small exporter countries; the expansion in Community, British and American potential has been below the world average.

As production potential has been rising faster than consumption, a considerable surplus has resulted, while at the same time import requirements have grown comparatively slowly. In consequence, idle capacity in the world overall, which in 1953 amounted to 16,500,000 ingot tons, was up by 1963 to 67,000,000 — a four-fold increase in ten years. Some of the countries which have expanded their potential the most are in addition notable for their low raw-material and labour costs. So it is not surprising that the share of Community exports in world trade in steel has slumped in the last ten years from 46 % to 32 %. This is, incidentally, a trend dating from before the war: a bare forty years ago the share of the six countries now forming the Community totalled no less than 70 %.

As the Community's exports have fallen its imports have risen. Its net exports — the excess of exports over imports — have dropped in the last four years alone from 12,000,000 to 7,800,000 tons, a decrease of 35 %. In contrast, net indirect exports — external trade in industrial finished products, expressed in ingot tons — have increased to 9,200,000 tons, thus outstripping net direct exports. Community exports are hence coming to be concentrated more and more on industrial finished products, and exports of rolled products of special quality grades. The ordinary steels which the emergent countries used to import, are now increasingly produced locally. And as these countries, by reason of their low raw-material and labour costs, are installing basic industries of their own, the products demanding less technical know-how are more and more being made there. From the expansion programmes announced, it must be expected that the coming years will see a further considerable increase in new production potential: that is to say, the changes, I have referred to, in the pattern of the world steel market will continue.

Intra-Community steel consumption, rather than exports of rolled products, is becoming more and more the mainstay of E.C.S.C. steel sales: as I have mentioned, the present revival is due almost entirely to the increase in internal requirements. But the trend in internal steel consumption too has been noticeably changing. In the first five years of E.C.S.C.'s existence it rose faster than total industrial production; since 1958 it has lagged behind it.

This change is due:

(a) firstly, to a relative deceleration in the growth of the most important steel-consuming sectors, and

(b) secondly, to lower specific consumption of steel.

Below-average growth is particularly apparent in the forges, the foundries, the shipbuilding industry, and various branches of mechanical engineering. In the transport sector, steel consumption has been disproportionately raised by the vigorous expansion in vehicle production, but this has failed to offset the relative shrinkage in the use of steel for permanent-way material and rolling-stock in consequence of which the total steel consumption of the sector has decreased from 25 % in 1913 to only 14 % in 1963. In contrast, in the United States the transport sector still does take about one-quarter of the total output of rolled products: in the spacious American market steel consumption capacity is obviously greater than in our small, densely populated European countries.

Grouping the various manufacturing industries by main categories, we find, generally speaking, that steel consumption in the capital-goods industries and the transport sector falls with rising *per capita* consumption, though rising on the consumer-goods side. In the building trade and in steel construction work steel consumption is also declining, though it should gradually even up at about 25 % of total steel consumption.

The structural changes in the manufacturing industries have affected not only overall steel consumption, but also the breakdown by types of product. Thus since the completion of the railway network production of permanent-way material has markedly decreased, while production of motor-body sheet has risen enormously.

The breakdown of steel consumption by types of products has also been altered by quality improvements to the steel themselves and advances in processing techniques. For example, the spread of automated welding has led to the use for various purposes of heavy and medium plate instead of sections. This change in the pattern of demand is, however, a very much smaller affair than the major switch in the transport sector.

Alongside these developments goes the reduction in specific consumption of steel, which is affecting the steel industry's sales outlets. The steel imput rate has long been falling steadily in a number of sectors, principally as a result of

(a) production of lighter-weight sections;

(b) production of steels of higher and more regular quality;

(c) more rational steel utilization;

(d) the substitution of alternative materials for steel;

(e) technical progress in other directions, leading to better performance for the same, or a smaller, amount of steel (*e.g.* more efficient utilization of energy and motor fuels).

The steel industry is thus itself in part responsible for the reduction in specific consumption, since it has been making its products lighter in weight and better in quality; on the other hand, these improvements have helped to create new sales outlets for steel in a number of markets.

The reduction in specific consumption can best be indicated by a few examples. It takes 20-30 % less steel to make one square metre of road bridge than it did fifteen years ago. Since 1950 the amount of steel going into the construction of a steam boiler with a useful steam rating of 30 tons has been cut by about 20 %.

Specific consumption of steel is also affected by changes in the size of the actual objects produced from it. For instance, the increasing popularity of the small car has brought the consumption per private car of rolled steel and castings down in Germany from 1,300 kg. in 1950 to only 1,000 in 1963. On the other hand, in the case of objects which are being built bigger and bigger, such as boilers, specific steel consumption is also being reduced. Any amount of similar examples could be quoted.

As regards the use of rival materials in place of steel, it is dificult to give precise details. Calculations to date indicate that slightly over 3 % of steel consumption is at present being lost to aluminium and plastics, and a further 3 % to concrete, timber and eternite — a total of from three and a half to four million ingot tons. It must, however, be borne in mind that new uses for steel have been opened up by its employment in combination with plastics. One important instance of the ousting of steel by a competing material is the preference now given to concrete construction in fields which used to be very largely the preserve of steel. The steel industry having succeeded in producing steels with very high upper yield points, it became possible by using these to build large-span concrete structures more cheaply than their all-steel counterparts. However, here the distinction is no longer clear between actual displacement of steel by its competitor and technical progress in the quality of the steel material supplied.

There are various ways by which the steel industry can check or offset the shrinkage in steel consumption. One is technological improvement:

(a) by making products light in weight,

(b) developing steel with better mechanical and metallurgical properties, and

(c) extending the range of products.

It would take too long to enumerate all the advances achieved in these respects in the last few years. It will give only one example, in the development of which the High Authority itself played a part. Some years ago a working party was set up, with High Authority co-operation, which in 1957 devised a new series of beams, the IPE beams, now progressively taking the place of sections that have in some cases been used for many years. As compared with the older products, the IPE beams represent a 7 % saving in weight on height of section, and a 22 % saving on modulus.

Considerable strides have been and are being made in the improvement of mechanical and metallurgical characteristics.

At the same time, if the steelmakers are to turn out products thinner in size and of better quality mechanically and metallurgically, improvements are needed not only in existing steelmaking methods but also in manufacturing methods. Since on the other hand it is important not to overstep certain limits in this regard, this necessitates co-ordination between the conditions the steel consumer requires his steel to fulfil and what the steelmaking techniques actually can perform. To help assure this the High Authority is issuing its series of Euronorms, drawn up in co-operation with producers and consumers, in which uniform technical characteristics are laid down.

At the same time work is going ahead on the technical processes involved at the manufacturing end, for instance on welding.

The marketing of steel will in the future be no longer a matter purely for the market operator and publicity man, but also more and more a matter of technical counselling. Just as the manufacturer's designs are adapted to the form and quality of the steel products, to ensure that these are employed to the best advantage, so the steelmaker too will compare notes on technical aspects with the manufacturer to obtain indications as to the requirements his products should fulfil. So there will have to be progress on both fronts, production and manufacture. This cannot be achieved without intensive co-operation by all concerned. In a number of cases some progress has already been made, as for example in the use of steel for motorway guiding-kerbs, in the facing of walls with plastic-clad or stainless steel sheet, and in the manufacture of furniture and fittings from steel instead of wood.

The promotion of steel consumption requires not only improvements on the technological side, but also new sales methods. As I say, this involves market study and the assessment of the manufacturers' experience with steel, but it involves more than that: what really counts, in making steel competitive from the manufacturer's point of view, is, as ever, whether steel can be supplied to him more cheaply than other materials. In a competition-based economy, the decisive consideration is, in the final analysis, price. And that is not just a facile simplification.

In the calculation that leads the manufacturer to opt for steel, he is of course influenced by the quality of the material and the cost of handling it, but the end purchaser of the product thinks also of the maintenance costs. Also, at the steelmaking end proper, they are considering new sales methods to secure a rationalization making for maximum efficiency in the utilization of rolling capacity.

The object for the future must be to work for parallel technological progress on the steelmaking and manufacturing sides, to seek out new uses for steel, and to publicize the lessons learned to as wide a circle as possible. This involves co-operation by all — producers, research centres, engineers and architects, and the public authorities. It was to foster such co-operation that the High Authority convened this Congress, and the subject "Progress in Steel Construction Work" was chosen because the building trade and steel construction absorb something like one-quarter of the steel produced, and because the technical problems involved in the utilization of steel for these purposes are of specially obvious economic importance.

The Congress, then, is intended to direct attention to the new angles of approach to matters connected with the steel market and steel utilization.

Jean-Marcel JEANNENEY

Chairman of the Congress

This is a technical Congress. The papers and debates fall primarily within the purview of engineers and architects. But in offering the Chair to an economist the High Authority has sought to show that it is not departing from what is, by rule, its own great concern — that of raising the standard of living in the member States.

Technology trills by its many-sided potentialities. It bears witness to the mastery man is gaining over nature. For thousands of years all he managed to do was to wield a few implements, domesticate a few species, smelt a few ores — and now in the last two centuries, and more marvellously yet in the last twenty years, he has been striving with passion to subjugate nature to his service. He is working to fathom all the mysteries — to penetrate the infinitesimal and to explore outer space. He is thinking up ways of bringing into play all possible manner of forces and materials. And he is contriving so to interconnect them that the means of production and the centres of production are changing from decade to decade with sometimes bewildering rapidity.

All this would be purely academic juggling with thoughts and with things, proof of the intellectual genius of the few, but valueless and indeed harmful to the many, were research not sensibly directed and the new techniques organized to serve, as Article 2 of the Treaty of Rome, confirming and supplementing Article 2 of the Treaty of Paris, well puts it, "to promote throughout the Community a harmonious development of economic activities, a continuous and balanced expansion, an increased stability, [and] an accelerated raising of the standard of living."

It was with this end in view that the High Authority decided to convene this Congress on Steel Utilization. A congress of technical experts, yes, but of experts met for the benefit of the iron and steel sector, and hence of the economy as a whole, so great to it is the importance of steel. Consequently, the implications of this occasion go far beyond the actual matters to be discussed during these three days. Taking this particular opportunity, the High Authority of E.C.S.C. is demonstrating that it is the duty of the European Institutions to ensure that technical innovations shall be a source of prosperity and not of disturbance.

That innovation can bring prosperity is clearly apparent, in Europe, in the United States and elsewhere. Quite manifestly, it is increasing the productivity of human effort. To reduce the demands on men themselves, it is drawing more extensively on natural energy; it is rendering production processes now more intricate, now more straightforward; it is even enabling work to be machine-done which until very recently seemed to be for ever possible only to the human brain — calculations, preparation of decisions, involving powers of memory and reasoning previously supposed to be found only in thinking beings. Again, innovation creates new employment opportunities, by the capital investment it induces and the requirements it brings in its train. It gives an impetus to the economy, it whets the spirit of inquiry, and by example stimulates inventiveness.

But what a mass of problems it produces too, by the resulting abrupt and unexpected changes in production conditions! Expensive equipment becomes suddenly obsolete; areas that led the field for particular products are left standing; workmen who have acquired their skills by long apprenticeship find themselves out of a job. The hardships caused by these upheavals never find their way into books. But untallied losses and untalliable hardships are losses and hardships just the same.

Sometimes, in face of the deluge of problems posed by technical progress, one does rather wonder whether it would not be possible to have prosperity without innovation. It is not a totally unreasonable thought. Once a given level of technical knowledge was reached, a people could live happy, with a rising population and a rising standard of living, simply by steady work in orderly, peaceful conditions, each generation taking over from its predecessors in accordance with the lessons of tradition. Enrichment would be slow, but the general well-being would be untroubled. In the Age of Enlightenment, European philosophers conceived of China as such a society, and would have wished to hold her up as the example for the enlightened despots of eighteenth-century Europe, whose mentors they aspired to be. But if the choice was ever there, for us it is there no longer, bitten as we are with the urge for the new and strange. Our thirst for knowledge is too strong, our concentration on greater affluence too rooted. In our time, the concept of progress has not, certainly, been reduced to purely material terms : it retains its grandeur, but technology has become its indispensable foundation.We are compelled to innovate, for to us stagnation would be a disgrace and a decay.

Inevitably, then, we have to accept that scientists and engineers should go on from discovery to discovery, indeed we have to urge them on. But at least we can try to ensure more harmony in their labours — first and foremost, so far as possible, by reducing under-employment of men and material. Unemployment and short-time working are an obvious evil, because they are painful; under-utilization of installations and equipment, which so often accompany them, are like them wasteful.

The temptation in any policy is to try to remedy matters by restricting the competition offered to the old by the new. This is sometimes allowable and even necessary, but over-reliance on it is liable to produce the airless deadness of a closed room. How much better to fight the adverse effects of innovation by innovating still more! So European steel is coming up against competition from steel made in other parts of the world, by new producers? Its customers are being offered concrete, aluminium, plastics instead? It is not a trend we must oppose, not even object to — provided the competition is fair — but at the same time we must not simply fold our hands and let the blast-furnaces and the rolling-mills grind to a halt. We must get the experts to engage in the necessary research so that the equipment we have, evolved by immense labours in the past, will go on doing its job in the future, whether thanks to improvements that will mean it does not have to be scrapped, or to new uses devised for its products.

In this age of ours, entirely new industries — electronics, plastics, nuclear production — are developing and flourishing spectacularly. Does their emergence mean that the older industries are doomed to decline? An industry is not a human being: the years do not make it old in the sense of "senescent." All industries are alike mortal, but their age is of no account. Their future depends on the perceptiveness of those at their head in looking into the future and settling for reasonable objectives, and their quickness in seizing on the opportunities afforded by inventions not directly connected with them. Still more, it depends on these men's readiness themselves to promote situation-saving inventions, not merely in order to use them in their own enterprises, but also to make them available to their suppliers, who will thus be able to supply them better or more cheaply, or to their customers, who will prosper accordingly and pass back some of the benefit to them.

To this course, by organizing this International Congress on Steel Utilization, the High Authority is urging the iron and steel industry — three or four thousand years old and still young and strong!

2 *Papers on the General Problems*

CONTENTS

Functional and Aesthetic Trends in Architectural Design in Steel Construction

(Original text : German)

Hubert PETSCHNIGG

The very phrasing of my subject introduces a dichotomy. In discussing functional and aesthetic trends in architectural design in steel construction, I have to approach my basic topic, steel construction, from two angles which are, or can be, very different indeed; they ought to be indistinguishable, but they are not, at any rate not nowadays. In appraising the position and trends in the field of modern steel construction, we can, and indeed must consider the aesthetic and the functional sides separately. The architect today is faced with the fact that the basic homogeneity of architectural design which existed, say, a hundred years ago, has been steadily disappearing, and today occurs only as an exception. Some contemporary engravings of the Crystal Palace in London, built — or rather assembled — by Paxton for the Great Exhibition of 1851 in Hyde Park (1,2) show that in former times, the architect was capable, at least in theory, of producing a structure in which function and form were perfectly wedded: A design could be reckoned a success, only if it fitted completely and naturally into place in the world, and to do this it had to be both aesthetically and functionally "right." One of the principal reasons why this was possible — in theory, as I say — was that the architect had a much more comprehensive grasp of his craft than we even attempt to acquire today. He was — ideally — primarily a master builder; one who not only knew one particular craft from A to Z, but knew each detail of every stage in the construction process; one who never needed to call on other experts to assist him in making his building safe, pleasing and serviceable.

Photograph 3 shows the Burgtheater in Vienna, designed and built by Semper and Hasenauer in 1870-1888. Semper was able to watch how, out of purpose materials and production techniques, form would emerge and grow and take shape, until it stood forth as sheer art detached from mere matter. Much as we would like to do the same, we cannot. This may be unfortunate. However, to moan about it serves no purpose, and after all, even if the dichotomy I speak of is now a permanency, that is not to say that we architects have thrown in our hand: we have not done anything of the sort. It cannot be disputed that many buildings which are now being constructed are indeed a pleasure to look at and a pleasure to use; buildings which do very successfully marry beauty with utility. The secret of achieving such buildings lies in co-ordination, co-operation and concerted effort. Here are two examples, the Seagram Building, designed by Mies van der Rohe, and Lever House, New York, the work of Skidmore, Owings and Merrill (4,5).

We are living in a pluralistic world. But it is a world in which we need not sigh for a homogeneity that is dead and gone: we must see too the many-sided potentialities that its many-sided nature offers us. When an age offers such a variety of incitements to think and model and mould as does ours, the creative mind is compelled to refashion the heterogeneous mass into a new whole.

Of crouse, the creative mind too changes with the passage of time. Building is no longer purely the affair of the master builder, — the architect himself. Since the mid-nineteenth century his dominant, indeed often sovereign influence on structure has steadily declined : since the mid-nineteenth century — that is to say, since the birth of steel construction. The revolution wrought by the development of the structural steel framework in the last century destroyed the predominance of the architect. In those early days, the architect viewed with the strongest suspicion the engineer's soaring imagination and specialized training — a training which enabled him to devise entirely unprecedented structures of steel, and what was more to build them. (Thus the Halliday Building in San Francisco still shows traces of past influences). While the engineers were logically following this course — all kinds of outstanding examples spring to mind (6) — architects went on for many years building as their fathers and forefathers had built before them. The buildings that typify this period, with their eternal imitations of the great styles of the past, rarely show the slightest sign of a creative approach : they are for the most part purely conservative, in the derogatory sense of the term. (For example, the Vienna City Hall built by Schmidt at the end of the nineteenth century) (7). New, forward looking design developed, as it were, on the fringe. As with the Tower of Babel, the experts were at cross purposes : they were not speaking the same language, and their work — meaning, here, architecture as the work of them all — just stagnated.

There are exceptions. Gustave Eiffel, who built the thousand-foot-high Eiffel Tower for the Paris Exhibition, seventy-five years ago, and so demonstrated to all the world the tremendous potentialities of the new technique, was not only a creative engineer; he was also a builder, a leading expert on aerodynamics, as well as being an adroit financier. His pioneering spirit anticipated many subsequent developments — for instance, he had components prefabricated in series, thus shortening and simplifying the on-site operations. But he did not really start a trend in his own day (8).

Industry, then proliferating everywhere, preferred to have its workshops and factories built mostly by ordinary technicians. Technical requirements and technical functions were the sole criteria, and though later generations have learnt that the purely functional can have a beauty of its own, the engineers who set the imprint of the new industrial world on the landscape knew little — and cared less — about such things : to them what mattered was to build a structure that would serve its purpose. Meanwhile architets were still clinging to an ideal of beauty that no longer matched either the times or the new technical facilities of the times : their imitation Renaissance, pseudo-Gothic, and neo-Romanesque was if anything even worse than the purpose-bound hideousnesses perpetrated by the bustling engineers (9).

We do not need however to concern ourselves too much about the sins of the past, particularly as we all know the sins of our own time will probably not appear so very much less heinous to our descendants, who are certain — like all descendants — to know everything much better than their forebears. However, I think it is fair to say that we have learnt a certain amount — and when I say " we, " I mean both architects and engineers. We know, of course, that the homogeneity of earlier centuries has gone for ever. But at least it is now possible for architects and engineers to do a creative job together. To that extent, in fact, it is not of such very great moment that I should have to discuss the aesthetic and the functional aspects of modern constructional steelwork separately. We have worked out how to live with our dichotomy, to learn from it and to use it as a source of creative innovation (10, 11, 12).

Every expert knows that modern architecture is increasingly influenced by the engineer's approach. Whereas formerly, to make doubly and trebly sure of safety, it was the practice to build " heavy, " nowadays we understand in much greater detail how to utilize the resistance of the materials, so that by planning in line with the material to be employed we can produce structures lighter in weight than would once have been thought possible. Of course, we must remember that neither heaviness nor lightness is an end in itself. One can run up a shanty town faster than one can build one decent dwelling-house.

The great point is that the building materials and the methods evolved by modern technology must be used to the best advantage. Steel remains one of these modern materials : it can be put to so many purposes that the architect is always finding new ways of bringing it in. Since the war the European steel industry has been supplying components on a massive scale more especially for industrial buildings. I must say I feel not enough has been done to work out methods for multi-storey structural steel buildings (13, 14).

As a result, the use of reinforced concrete for superstructures has become still more common, especially as this method has been regarded as the natural development from the traditional solid-stone building : it could be carried out by the old type of craftsman, and the planning system too did need to be much altered, changes during building being quite feasible with reinforced concrete. The architect also frequently did not feel quite happy with the meticulously exact planning required for steel construction, and preferred to stick to the apparently easier method with reinforced concrete.

Structural steel framework is quite different. It is less " conservative ", and it certainly needs exceedingly accurate and time consuming advance planning; it takes months for the architect and engineer between them to finalize in detail the core of a multi-storey building. But it offers a number of possibilities which I feel to be of considerable importance for our own time and for the future (15, 16, 17).

The reason why many architects are so dubious about building in steel is undoubtedly partly that they just have not got the hang of this long, detailed planning. I will venture to suggest in this connection, that the universities and technical colleges might make more of a point of training their architectural students in the necessary fundamentals. Incidentally, our hosts, the High Authority, could do a very valuable job by promoting this in the Community countries.

Structural steel framework of course is only one of the possibilities in the building field today, and its pros and cons must be weighed up against those of other methods. Initially, high building costs are something of a deterrent. What really matters however, is not the cost of the shell as such, but the total cost of the completed building. If provision can be made in the load-bearing structure for the finishing stages, the result is a saving in time and money. By its very accuracy, structural steel is ideal for the preparatory work for the finishing stages : **precision of measurement is the key to prefabricated industrial building**. It must also be borne in mind that industrial fabrication of components has a considerable advantage in the present economic situation of manpower shortage and spiralling costs of labour and materials. Restrictions on the space available at the site can also be an argument for prefabricated components.

Considerations of weight are not very important. The lightest structure is not necessarily the most economic and serviceable, and the more conservative building methods will certainly continue for some time to present various advantages which may be preferred by some clients.

Life can appear quite different seen from the new lightweight structures. In these thin-walled buildings of steel and glass one is more exposed to external influences than behind thick, chill- and warmth-retaining masonry. On the other hand, greater expenditure of technical skill and know-how is involved.

Speaking very generally, building can be said during the last few years and the last few decades to have moved away from the solid stone or brickwork style in the direction of the structural framework. It has on occasion been pointed out that a comparable trend is observable in Gothic (18), that the human spirit always feels the urge to master matter, and — the argument runs — structural-framework building gives it a change to do just that. I do not feel, myself, that historical parallels of this kind are very illuminating, particularly as it tends to be forgotten that the apogee of Gothic was comparatively brief: its vitality soon evaporated more and more, until ultimately it gave place to the very solid, earthly, material building of the Renaissance and later the equally material Baroque; as example: the Residenz in Würzburg, built by Balthazar Neumann in 1720) (19).

My own opinion is that the trend towards lightweight structural-framework building is one that is essentially bound up with the techniques of our time. In the United States it is already further advanced; Europe, once more, is following the American lead, with a certain timelag. This particular timelag seems oddly persistant. It might have been supposed that in the present stage of technical progress, with the constant exchange of periodicals and books and with ease of travel, the latest experience would be readily passed on and taken over from continent to continent. But that is not happening. We just have to accept the age-old truth that man proceeds by trial and error. Only by actually planning and erecting buildings ourselves can we really learn for ourselves what we need to know. Clients may groan, but I am afraid the fact remains that mistakes continue to occur which five or ten years later look almost incredible. The trend goes on — not with any very notable regularity, but still it goes on, both over here and across the Atlantic (20-35).

In practice, there have been a number of recent new developments which open up fresh possibilities. New types of sections for constructional steelwork have been and are being developed, and the former practice of utilizing the metal to the extreme limit, by the use of fabricated compound sections, is being progressively discarded, The Euronorm sections make for substantial savings in weight, and for simple joints and connections. It is being found an advantage for economic serial building to maintain the cross-sections of supporting members throughout the height of the building. The Euronorm range of sections of various flange and web thicknesses forms the basis for this; at present, however, the adoption of this method is being impeded by the long delivery dates.

Major new possibilities are also emerging thanks to the freedom of choice in the shaping of cold-rolled sections. These form the basis for a new type of light-weight steel building whose development is just beginning (36-45).

We stand at the beginning. Such advances in structural steel building as have been achieved are no more than rungs, of varying importance, on the way up the ladder. I should like just to mention one or two constructions for which I have myself been responsible, and which I am now only too well aware were deficient in certain respects (46, 47).

The clients concerned must forgive me for taking such a critical view of my, and so of their, buildings. It is however a long hard road to perfection, and I do not think there has ever been an architect who has arrived right at the end of it. Before this audience of international experts, we must discuss both the credit and the debit side with complete frankness if we are to form a realistic picture of the present state and future prospects of constructional steelwork — and unfortunately past experiences are apt to be painful in parts.

And so to my summing-up. You may possibly feel I have concentrated too much on function and purpose, and not said enough about aesthetics, line and form. But function and purpose and building practice, to my mind, are the basic prerequisites which have to be grasped before we can go on to deal with the aesthetic aspects. And this brings me back to the dichotomy I referred to at the beginning : function and form no longer coincide of their own accord, they have to be aligned and co-ordinated. That the focus should be on function may perhaps distress those who consider the noblest attribute of a structure to be elegance of line. But surely it would be rather starry-eyed of us to yearn, in this sober, purpose-directed world of ours, for shapely buildings little if at all fitted to their workaday object. We have to accept it — though I dare say it is badly expressed — that the function of the structure comes first. Perhaps — I am sure I hope so — there is no real question of first or second. Perhaps it is still possible, or again possible, today that there are basic forms of architecture which the architect and engineer must seek together and constantly rediscover.

But in practice the fact remains that the functional side has to be considered first : it is on the basis of its function that an aesthetically pleasing structure must be evolved. To this consideration we owe the very marked influence of the " engineer " approach on modern building. But to it we also owe the tremendous opportunity to use the new means and methods to produce something really new in itself — a new style in building, stemming from intensive co-operation between architect and engineer, or, if you like, between engineer and architect. The new style must and will of itself beget a building aesthetic. And I think that signs of this new aesthetic are already apparent to all those with eyes to see.

Description of photographs

1 - 2 — Crystal Palace, London 1851.

3 — Burgtheater Vienna, 1870-1888.

4 — Seagram Building, New York.

5 — Lever House, New York.

6 — Golden Gate Bridge, San Francisco.

7 — City Hall, Vienna.

8 — Eiffel Tower, Paris.

9 — View of colliery buildings, Salzgitter.

10 — Multi-storey steelframe building, New York.

11 — View over East River of Manhattan and U.N.O. Building.

12 — Chase Manhattan Bank, New York.

13 — Steel construction of Technical Institute, Ludwigshafen.

14 — Pirelli Building, Mailand.

15 a - 15 b — Mannesmann Building, Düsseldorf.

16 — Inland Steel Building, Chicago.

17 — Dining Hall of Bayer Werke, Ürdingen.

18 — Reims Cathedral.

19 — Central Pavilion of Würzburg Castle.

20 - 21 — Evangelical Church, Leverkusen-Bürrig; interior view and roof construction.

22 - 25 — Phoenix-Rheinrohr Building, Düsseldorf; general view, south view, floor plan and roof detail.

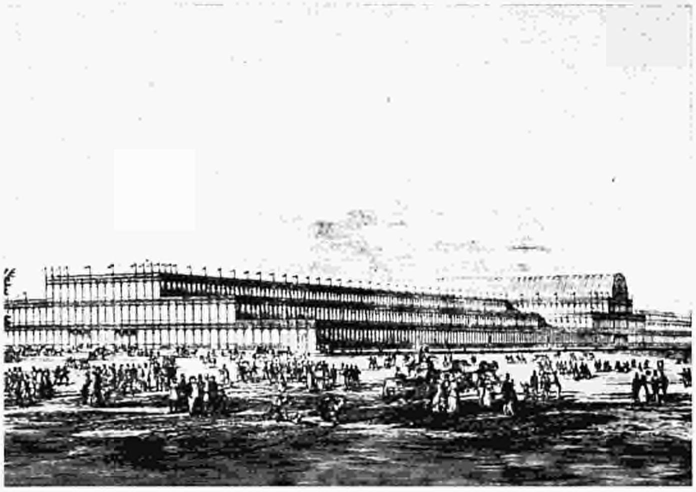
26 - 29 — Bayer Building, Leverkusen; general view, floor plan, steel skeleton and north view.

30 - 35 — Unilever Building, Hamburg; general view, steel skeleton, roof construction detail, façade, steel skeleton.

36 - 45 — Bochum University: floor plan, prefabrication of sections (two views), filling sections with concrete, hardening of concrete, transport and posing of sections, corner construction, view of finished floor.

46 — Pampelfort Office Building, Düsseldorf.

47 — Lake Shore Drive, Chicago.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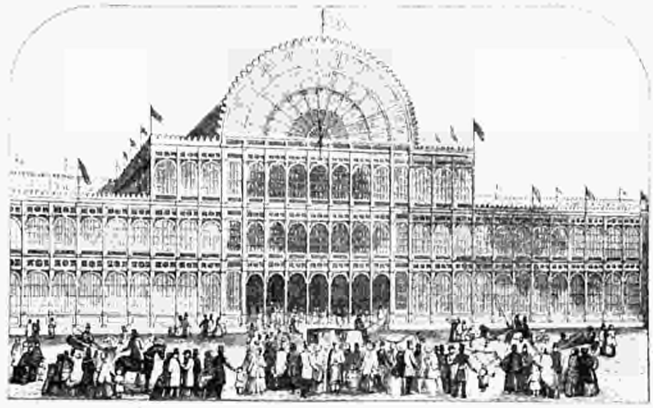
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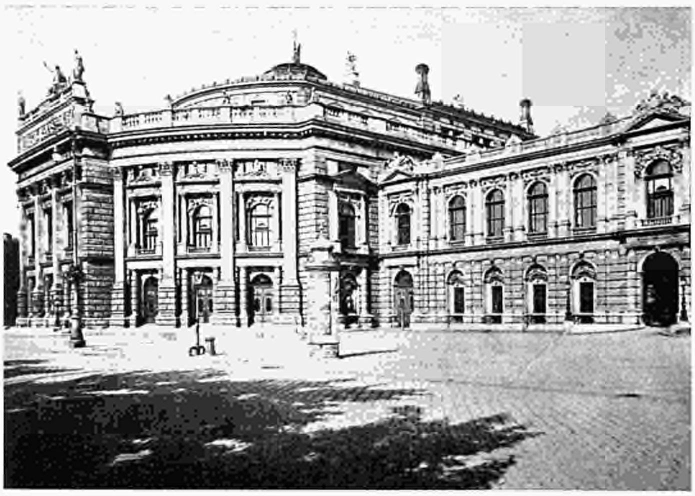
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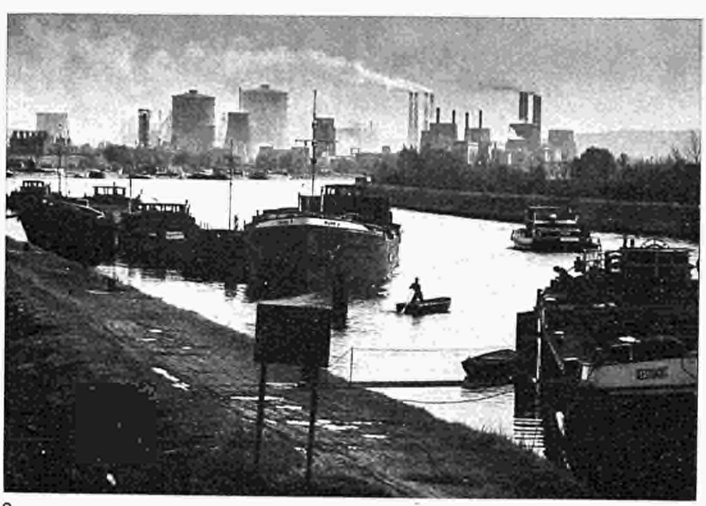
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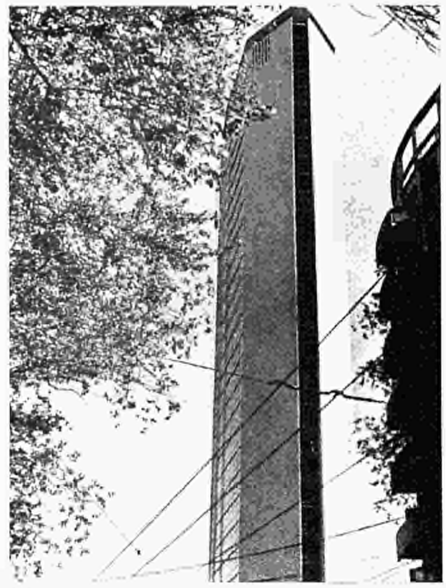
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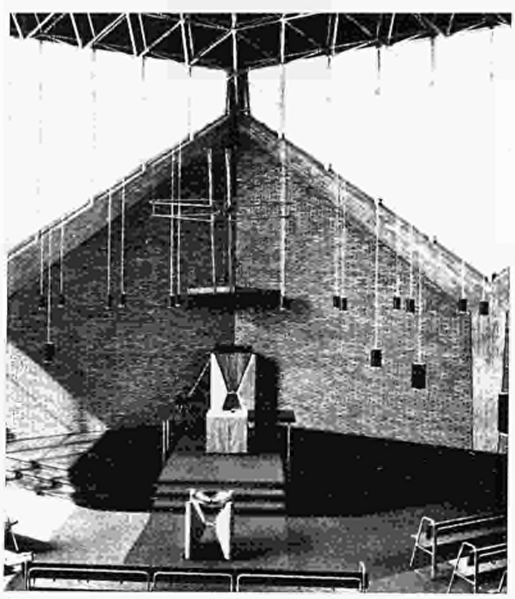

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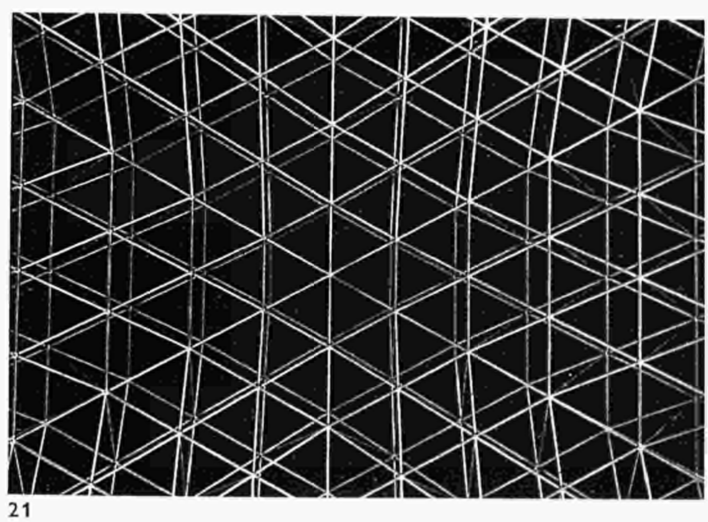

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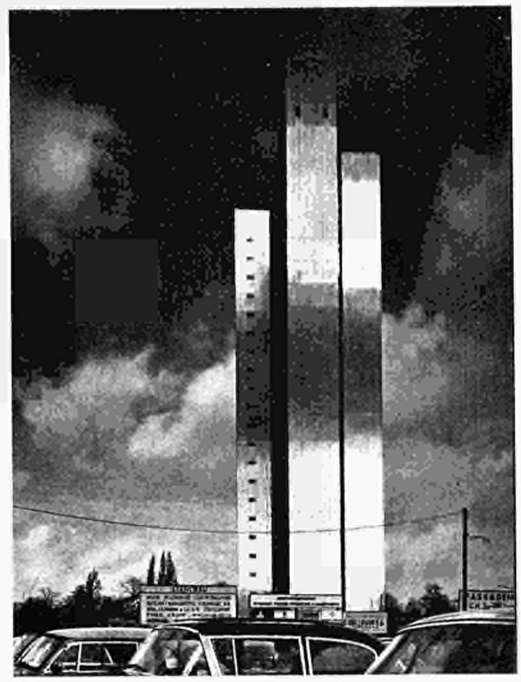

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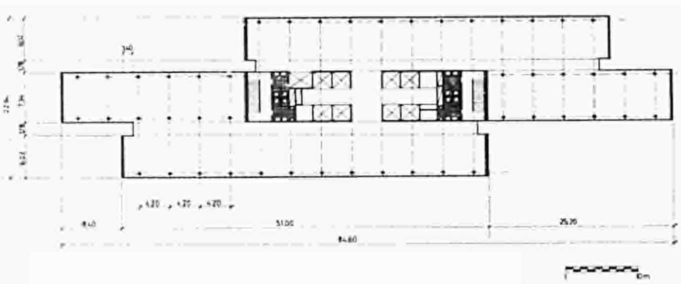

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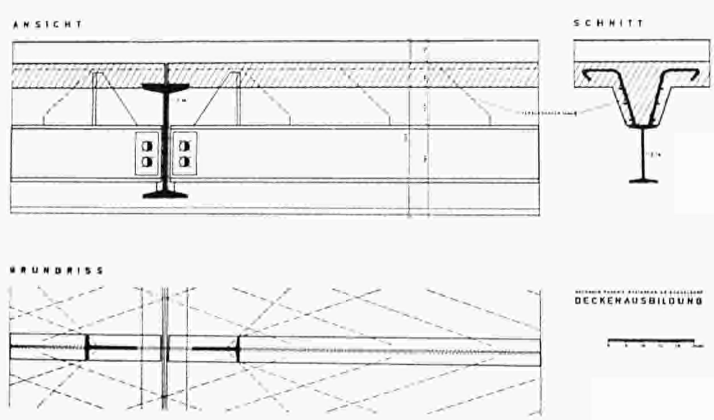

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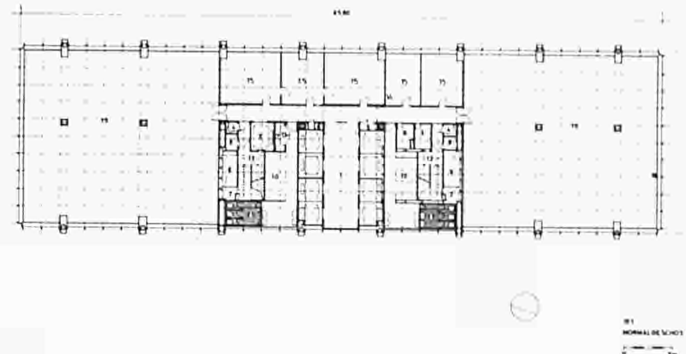

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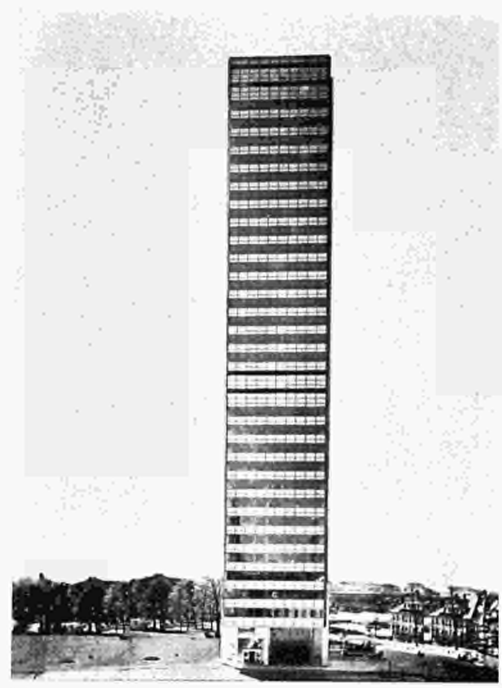

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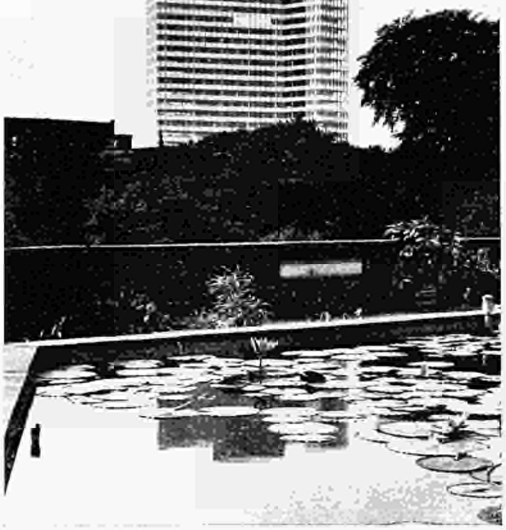

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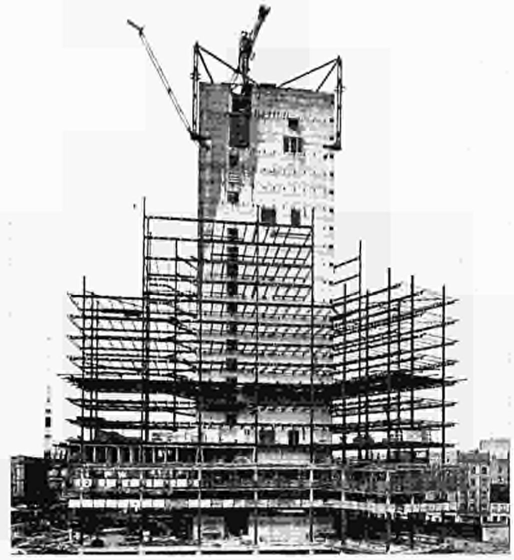

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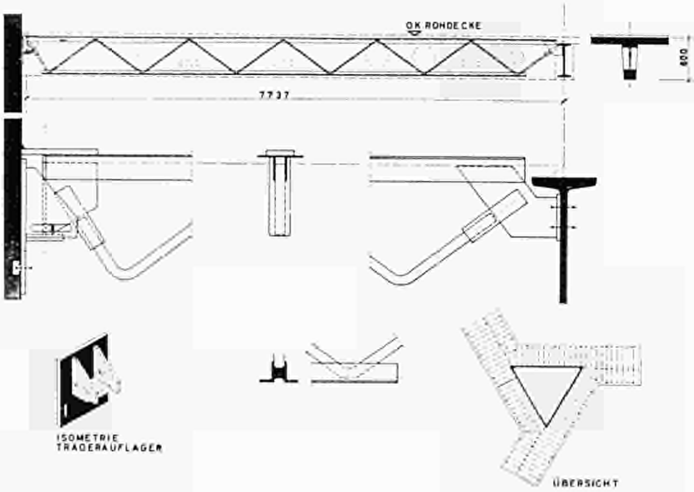

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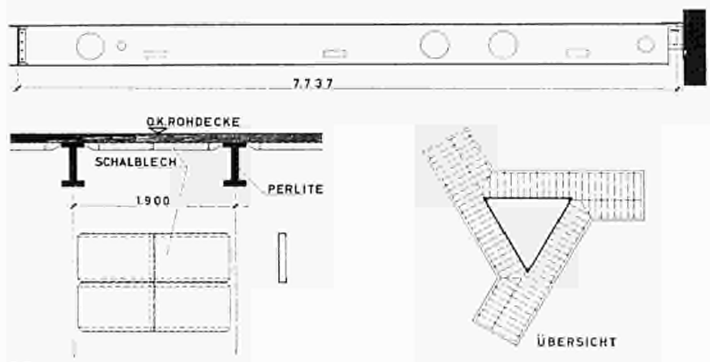

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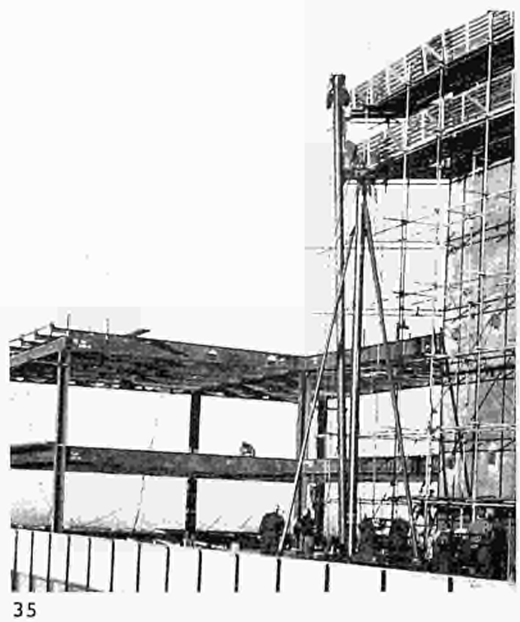

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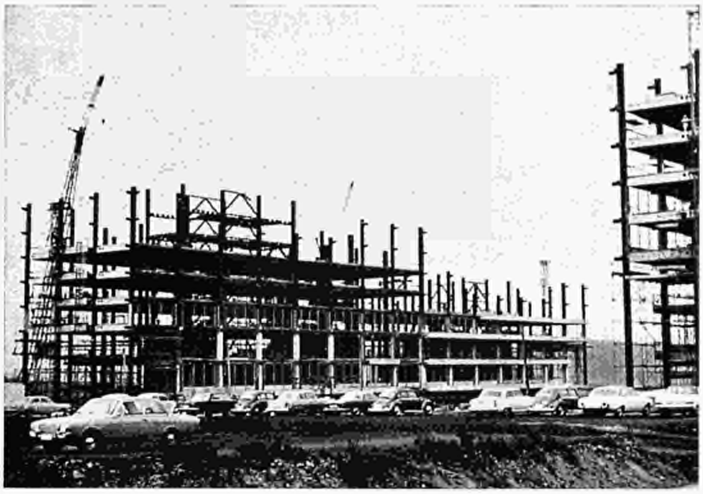
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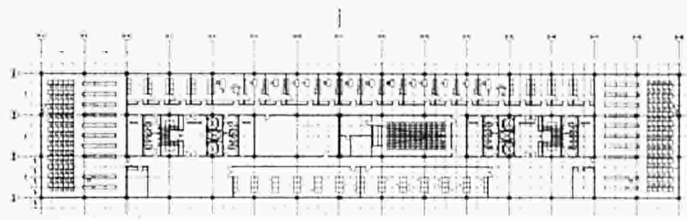
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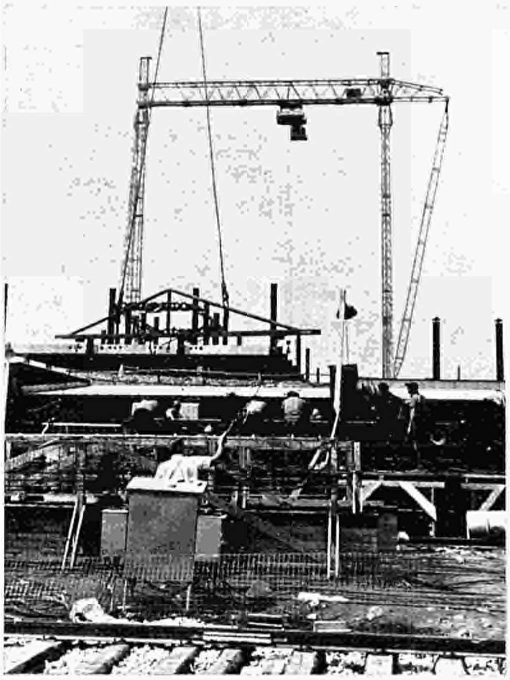
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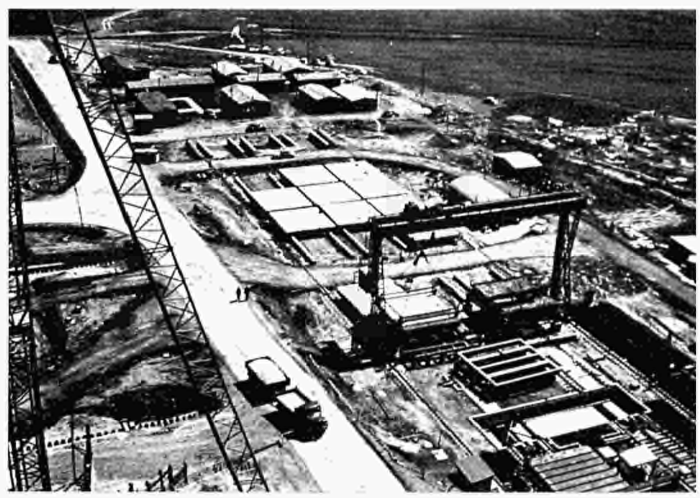
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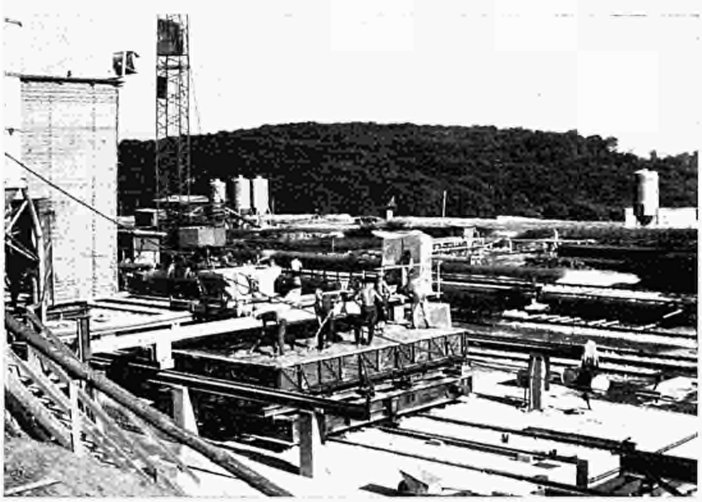
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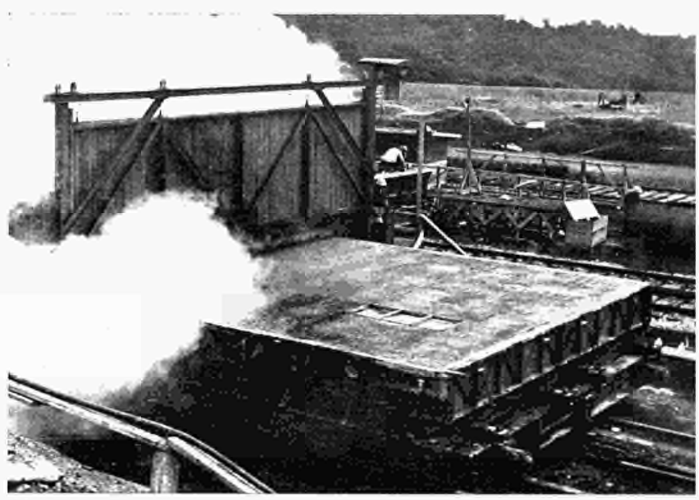
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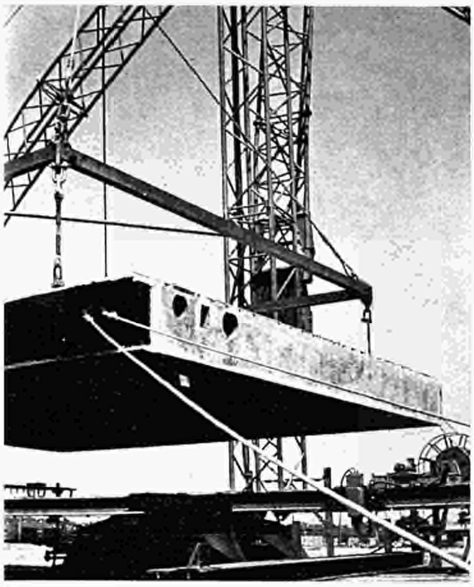
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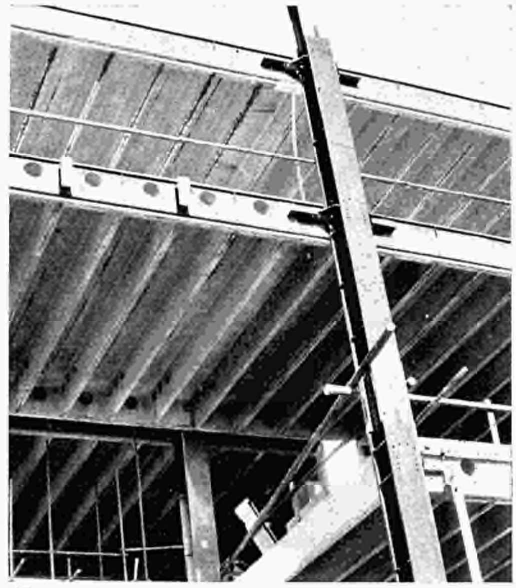
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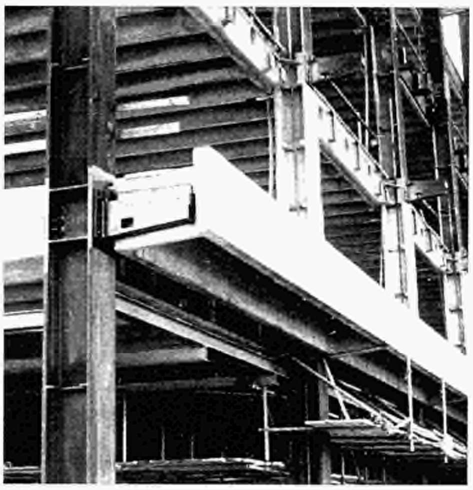
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Present Position and Future Developments in Steel Construction Work

(Original text : German)

Fritz STÜSSI

In order to be able to judge the present state of development of steel construction, it is necessary, by comparing representative structures of former days with those of the present time, to ascertain the general features of this development and the means employed to achieve it. Steel construction is a relatively young method of construction. It was not until the end of the 18th century that it began to compete with the earlier materials stone and timber, and for this reason its development to date is still comparatively easy to trace. We shall confine ourselves to the most representative categories of steel structures; bridges, steel-framed buildings and shed-type buildings. Steel construction does, it is true, comprise other important fields of application, such as steel hydraulic engineering construction, the construction of pylons for power transmission lines, lifting appliances, steelwork tower structures, etc. However, lack of time, if no other reason, compels us to confine our attention to the above-mentioned three main categories which should be sufficient for our purpose.

With regard to the construction material, the evolution can be characterised by considering briefly some important structures. In 1777-1779, Abraham Darby built the Severn Bridge near Coalbrookdale, a cast-iron arch bridge of 100 ft span (1). This first structure built in this new material already shows two characteristic features; in its external shape it resembles the stone arch bridges, but the individual structural components are of open (pierced) construction and thus already exhibit the particular feature of lightweight construction. Furthermore, they were fabricated in a workshop and then assembled on the site. They thus show the subdivision — typical of steel construction — into shop work and erection work, and therefore also present the distinctive feature of prefabrication of the components, which is something with which reinforced concrete and prestressed concrete are so greatly preoccupied at the present time.

In 1816, the Frenchman Seguin was the first to use wires for building fairly small suspension bridges. It is more particularly this form of construction, as we shall see later on, that has led to the present great achievements in the erection of longspan bridges.

In 1820-1826, Thomas Telford built his famous suspension bridge across the Menai Straits with wrought-iron chains using a 177 m long main span (2). This structure is of fundamental importance to the development of bridge construction and, indeed, to that of constructional engineering as a whole for the further reason that it provided Louis Navier with the stimulus to write his first important work "Rapport et Mémoire sur les Ponts suspendus", Paris 1923. It embodied the first scientific investigation of a structural system and, together with "Résumé des Leçons sur l'Application de la Mécanique", which appeared three years later, it constitutes the actual basis of present-day Theory of Structures and was of comprehensive enough to tackle the structural problems of those days. The Theory of Structures, for which the foundations were thus laid, is an applied science. It exists solely in connection with the structures to be designed with its aid. Thus it is a means that the design engineer calls to his assistance, and it is important always clearly to realise the essential fact that structural analysis is not in itself a creative activity, as it cannot design buildings and structures. Its task consists merely in verifying the strength of existing structures or of structures as yet existing in the designer's mind.

Obviously, it is of fundamental importance that our structures should possess adequate safety and that they should be reliably calculated to ensure this. A design which is not based on a detailed and realistic set of calculations will never produce a high-quality structure.

With his Britannia Bridge, a wrought-iron solid-webbed girder bridge with a maximum span of about 142 m (3), Robert Stephenson introduced wrought iron into bridge construction. This bridge, too, which was built in 1844-1850, was of particular importance with regard to the evolution of scientific thought: the exhaustive and fundamental tests for clarifying the stability problems associated with the thin-walled steelwork components introduced true experimental research into the service of the direct design of a structure.

Finally, in the construction of the bridge over the Vistula near Fordon in 1890 (4), mild steel made its entry into structural engineering, the designer of this bridge, Georg Christoph Mehrtens, having first satisfied himself of the equivalence of the acid and the basic method of manufacture of this steel. As long as structural steelwork was constructed with riveted connections, the designer did not inquire about the method of manufacture of the steel. The decision was left to the steelworks, and the choice of manufacturing process depended on the characteristics of the cast iron to be remelted or of the available iron ores. Differences between the three quantitatively most important steelmaking processess — Bessemer, basic-Bessemer and open-hearth — did not assume importance until welding was introduced, more particularly in connection with the demands as to "weldability" that came to be applied after certain serious mishaps had shown this to be necessary.

As a background to this brief outline of the evolution of steel construction let us now consider some remarkable present-day structures. Foremost among these is the George Washington Bridge over the Hudson River in New York (5), which was built by O.H. Ammann and opened to traffic in 1931. With its construction an old dream of engineers, namely, to build spans of more than 1,000 m, came true. About half a century earlier, Roebling had built the Brooklyn Bridge (1883) (6), a suspension bridge with wire cables and a span of about 500 m. He used cables composed of parallel wires having a tensile strength of 11.3 t/sq.cm. Hence O.H. Ammann doubled the attainable span at one stroke. The wires he used had a tensile strength of about 15.5 t/sq.cm, a value which, to date, can probably be regarded as the upper utilisable limit for cold-drawn wires for suspension bridge construction. A further increase in strength is in itself indeed possible, but this is associated with a reduction in the elongation at fracture, *i.e.*, in the ductility of the material, which is dangerous, because it can give rise to fracturing of the wires. The great step forward taken by O.H. Ammann therefore cannot be explained by the relatively modest increase in the strength of the wire employed. More important was the intellectual achievement, namely, the correct understanding of the forces and deformations in the heavily loaded cables such as occur in long-span suspension bridges. This understanding had to be backed up by calculations, and the construction of the George Washington Bridge accordingly called for the refinement of the deformation theory of suspension bridges founded by Joseph Melan and Wilhelm Ritter. Such refinement was no longer accurately possible with the functions made available by analytical theory. It thus becomes clear that further development of structural design methods, in addition to requiring analytical solutions, calls more particularly for the elaboration of numerical methods of calculation which can be adapted to the peculiarities of the pattern of forces acting in the structure to be built. In this connection the words of Pierre-Simon Laplace (1749-1827) may be of great significance with regard to structural engineering too: "*La nature se moque de nos difficultés analytiques*". The great importance of the George Washington Bridge with regard to the development of steel-bridge construction is very clearly revealed if we compare the earlier designs for bridging the Hudson River with one another. The need for a bridge existed as far back as the eighteen-eighties, and designs were prepared by prominent engineers[1] (Figs. 1, 2 and 3). None of these designs was carried out because none was able to provide a suitable technical or economic solution to the problem. The credit of achieving this goes to Othmar Ammann's design, which, as compared with all the earlier designs, embodies a new and decisively important feature in the evolution of steel construction, namely, the trend towards clear-cut and simple structural forms suited to the natural mode of functioning of the structure. The capacity of the George Washington Bridge has recently been much increased by the addition of a second deck (7), the structural components such as the cables, towers and anchorages having from the outset been designed for this increased loading. Taking the George Washington Bridge as our starting point, it is, with our present-day means, possible to built suspension bridges with still longer spans. The biggest suspension bridge now in existence, the Narrows Bridge in the New York harbour area (8), which has a span of 4260 ft (approx. 1300 m) and has likewise been built by Othmar Ammann, will be opened to traffic shortly.

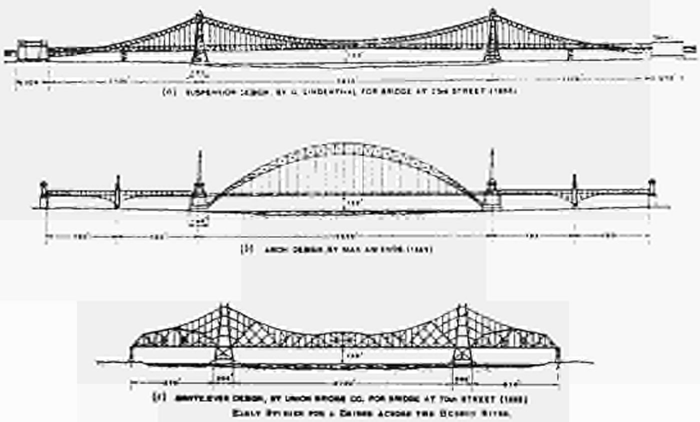

Fig. 1

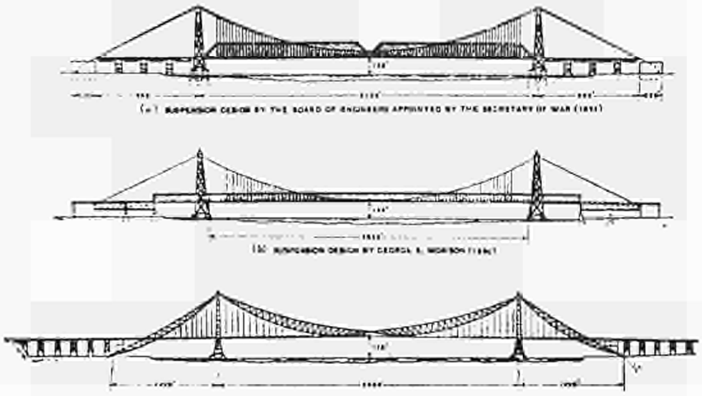

Fig. 2

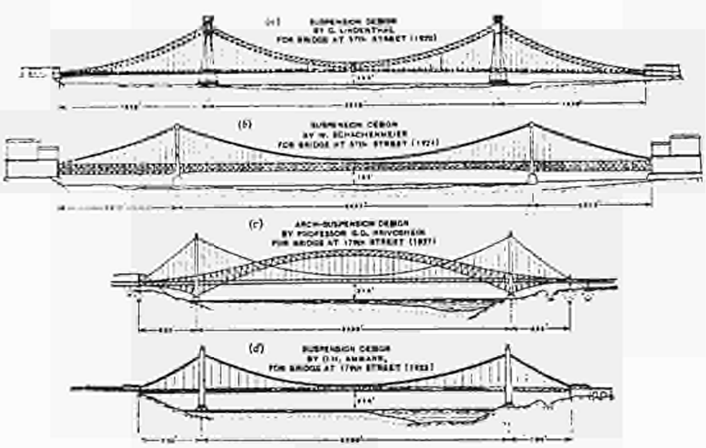

Fig. 3

It is noteworthy that particularly at the present time a number of major bridges are under construction, *e.g.*, over the Tagus at Lisbon, over the Severn in Britain, over the Orinoco in Venezuela; and recently, on 4 September, 1964, the new road bridge over the Firth of Forth, near Edinburgh (9), was ceremonially opened. The Forth Road Bridge constitutes a milestone in the development of European bridge-building, for it is the first bridge in Europe to have a span of more than 1,000 m, which goes to show that this continent's steel construction industry is now perfectly capable of solving even the biggest structural problems. A notable feature that calls for mention in connection with the Forth Bridge is that at no other bridge construction site had such high wind velocities hitherto needed to be taken into account, a fact which rendered increasingly difficult the work of erection. The cross section of the bridge gives a fundamental indication as to the solution of the problem of aerodynamic stability, at any rate for the completed bridge under service conditions (Fig. 4). During construction, the high wind forces, despite all the precautions to control them, caused a very considerable loss of working time. The new Forth Bridge is situated close to the railway bridge over the Firth of Forth, which was opened in 1890 and is one of the longest span lattice-girder cantilever bridges in the world. Thus, two masterpieces of the art of bridge building can be seen side by side, and together they represent an important period in the evolution of steel bridge building. The same evolution is perhaps even more clearly apparent from a comparison of the O.H. Ammann's Bronx-Whitestone Bridge (10), whose simplicity and elegance of structural form can hardly be surpassed, and a view of the interior of the Quebec Bridge (11), which was completed during the First World War (1917) and is the second of the great cantilever bridges. The trend towards simple, straightforward structural forms has become an essential feature of present day steel construction.

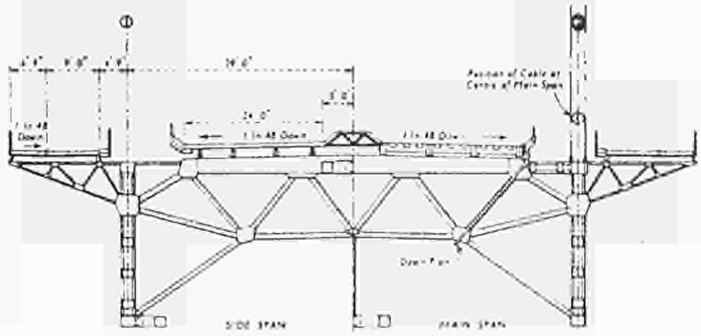

Fig. 4

In the domain of the construction of steel-framed buildings the steel structural framework predominates for buildings more than 125 m in height. For such structures the essential requirement is the use of the most efficient materials for the two functions concerned, namely, the structural (or load-bearing) function and the infilling (or cladding) function. For the structural system, steel is particularly suitable because it is a high-strength material which enables columns to be designed to relatively small cross sectional dimensions even in the lower storeys and thus necessitates only a relatively small reduction of the available effective floor space. This, in addition to the short construction times involved (interest on building capital), is a significant economic advantage of the steel structural framework. The tallest steel-framed building hitherto constructed is the Empire State Building in New York (12) with a height of 381 m (not including the aerial tower on top) and 102 storeys. This significantly exceeds the height of the Eiffel Tower (13), which is 75 years old this year.

Shed-type single-storey buildings show a great abundance of structural forms. Two fairly minor examples, the Palais des Expositions at Geneva (14) and a building for the Mustermesse at Basle (15), show the adaptability of steel construction to the state of the market at any particular time. The economy of a solution is determined by the total cost, which comprises the cost of materials and the cost of fabrication. As these two examples show, adaptation to the state of the market can be achieved by reducing the steel consumption in conjunction with increased fabrication cost or, alternatively, reducing the fabrication cost in conjunction with increased steel consumption. A problem which keeps cropping up in connection with buildings of this kind is the requisite degree of fire protection. By way of example, mention may be made of the steelwork of an aircraft hangar at Zurich airport (16), for which the fire inspection authorities at first insisted on complete fireproof encasement of the lattice-type roof trusses. It was then pointed out to them that a fire was hardly likely to start in the steelwork, but would develop in one of the aeroplanes in the hangar and that the value of one such plane was far higher than the cost of the steel structure as a whole. This argument at any rate produced an acceptable compromise in the form of a fireproof ceiling suspended from the bottom ties of the trusses. Admittedly, this did not improve the internal appearance of the building from the aesthetic point of view, but it did not excessively add to the cost.

The tasks that have to be tackled in present-day structural steelwork research are very numerous and varied. These will now be briefly outlined with regard to three main directions of research : material, structural connections, and design methods.

Lack of time makes it impossible to give anything like a detailed account of the influence of welding on the special requirements applicable to structural steel. However, the significance of the danger of fatigue under live loading will, at least, be briefly indicated. The tests recently carried out by the American Association of State Highway Officials (AASHO) clearly show that, with the present-day large traffic loads, a real fatigue danger exists, not only in the case of railway bridges, but also in that of road bridges [2] (Fig. 5). The results of these tests, in which incipient cracking and failure were found to occur after about only half a million load repetitions, are entirely in agreement with the results obtained in laboratory tests. It should particularly be noted that, for low minimum stresses, the fatigue limit may fall below the permissible stresses laid down in various Standards or Codes of Practice. This result calls for a more stringent formulation of the design rules with regard to fatigue. It should also be noted that, with inadequate design, fatigue phenomena are also liable to occur under wind pressure.

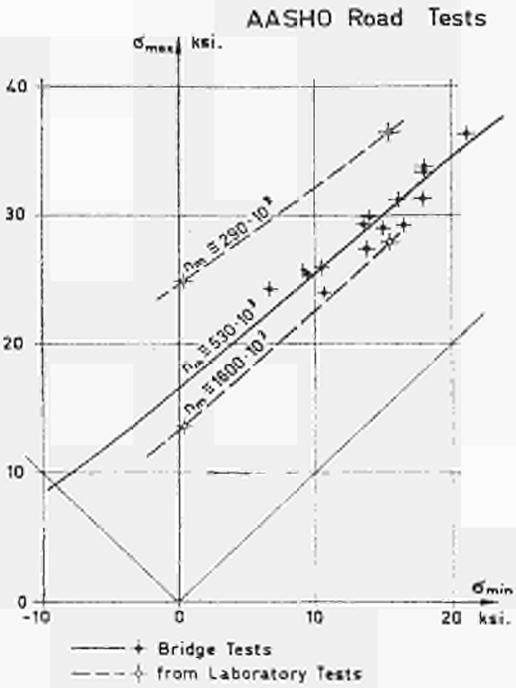

Fig. 5

In addition to riveting and welding, the two methods of connection hitherto employed, high-strength boting has increasingly come to be used. The high-strength bolt is in itself, of course, more expensive than the rivet, but a saving can be effected because it is much simpler to use than a rivet, so that the connection as a whole, in cases where small numbers of bolts are involved, can be formed more economically by bolting than by riveting. This advantage becomes more pronounced as the number of erection joints in a structure are smaller, and this is in line with a general trend of development in steel construction, namely, towards increasing the shop work as far as possible in order to minimise the amount of expensive erection work on the site. This calls for increased capacity of the lifting appliances and means of transport, on which the fundamental division into shop work and erection work largely depends. The use of adhesives for bonding in steel construction is still in its infancy, though already it is possible to use this method for connections which do not have to transmit any large forces. Further development of the use of adhesives is a task for the immediate future.

With reference to structural design methods, some fundamental aspects of the problem must at least be pointed out. The calculations for a structure are an integral part of the design and should therefore be within the range of ability of those engineers who design and work out a structure.

This means that the methods employed for the structural design calculations constitute the basis of the design of the structure in its various details. In their manner of representation they must therefore enable the designer to visualise them directly and they must be dependable and realistic. In many cases the method of design calculation based on analysis fail to fulfil these requirements. Thus, it is astonishing that, for in the case of so simple a structural element as a rectangular plate under uniformly distributed superimposed load, analysis can offer no functions that provide a straightforward soluriou of the differential equation of the bending of the plate. It is for this kind of problem that numerical methods are very important. Perhaps the most characteristic example of such methods is afforded by the funicular polygon, which in its basic conception traces its origins to Leonardo da Vinci and which, when translated into mathematical language by means of the funicular polygon equation, enables most of the differential equations occurring in structural design to be conveniently solved, including more particularly those cases where analysis fails to provide solutions.

As regards a first-class, up-to-date solution for a structural engineering problem, the requirement as to economy is of major importance. Nowadays the terms industrialisation, rationalisation and standardisation are frequently used in this connection. These represent requirements which are already largely fulfilled by present day steel construction. The separation of the construction work into the manufacture of the initial elements (rolled steel sections), shop work, and erection clearly shows the characteristics of a rational division of the overall work. Recently the High Authority of E.C.S.C. introduced new standards for the two principal forms of rolled steel sections, namely, the standard I-section and the broad flanged I-section. For each series of sections the essential thing is to find a solution that is acceptable both to the manufacturer and to the user (Fig. 6). The user would like to have closely graduated sizes of sections at his disposal, whereas the manufacturer, for the sake of economical production would like to confine himself to the least possible number of sizes. The present arrangement appears to be acceptable to both sides, as the increase in weight per lineal metre from one size of section to the next in both types corresponds to about 15%. In large structures a skilled designer can ensure that the steel sections are for the most part well utilised. However, it must be emphasised that the economy of the structure as a whole is much more dependent on whether or not the overall design is efficient than on whether or not the permissible stress can be fully utilised in some of the sections. What is important is that, in the case of broad-flange beams, the Differdange rolling mills will, if sufficiently large quantities are ordered, also supply intermediate sections. From the designer's point of view it is desirable that this possibility be retained, despite the overall standardisation that has been introduced.

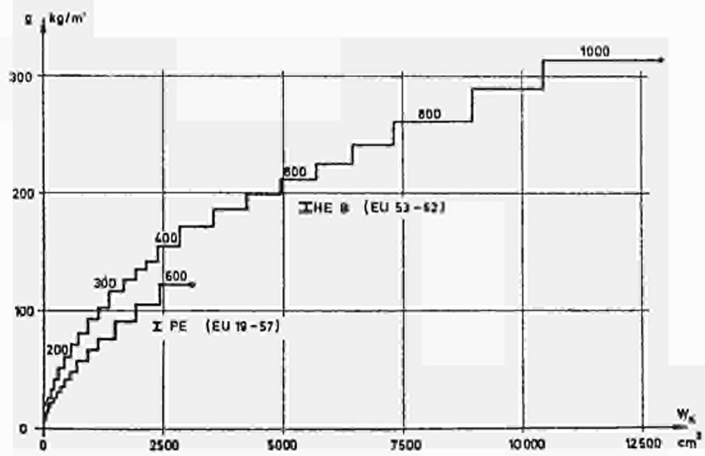

Fig. 6

Light-gauge cold-formed sections formed by folding are coming into increasingly widespread use. In this respect it must be noted that local buckling — because of the thin walls of these sections — is liable to occur at relatively low stresses. Failure of the structural member will, however, occur only in the supercritical range, and the design of light-gauge sections does, in fact, make due allowance for this. This means, however, that full utilisation of the permissible stresses in so longer possible. Instead, we must adopt so-called reduced widths in the calculations (b_r), so that the economy of the use of such light gauge sections is greatly restricted [3,4] (Fig. 7). These sections may be economically advantageous with relatively small loads if, in addition to their load-bearing function, they can be given another function to perform, e.g., for mezzanine floors in steel-framed construction.

The prerequisite for industrialisation is the manufacture of identical products in fairly large quantities. In steel construction this is, as a rule, possible only to a limited extent, for every structure has its own particular problems to solve. In a large country it would, for example, be possible to build schools of standardised types to the same plan throughout the country. However, it can safely be assumed that in Europe the wishes of the individual educational authorities cannot, for the present, be lumped together, ignoring all distinctions. Hence industrial mass production of this kind is likely to be severely limited in Europe for the time being.

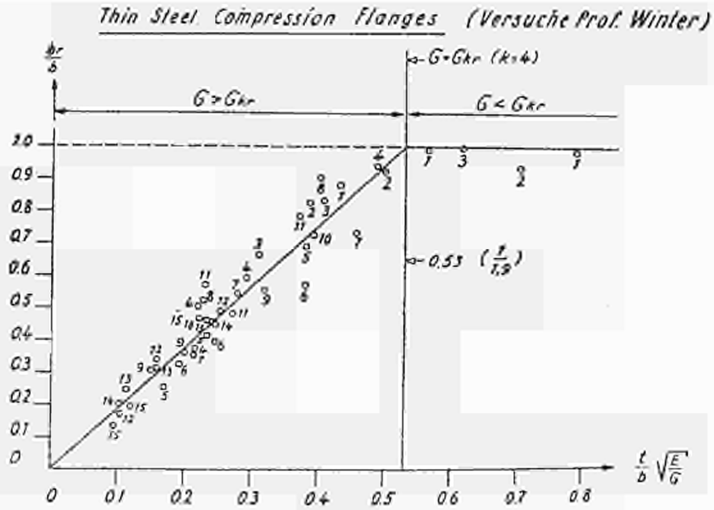

Fig. 7

A factor which may enter into the economical solution of a major design problem is the use of ordinary and high-tensile structural steel in one and the same structure. This is by no means a new feature in steel construction. Thus, as long ago as 1826, Louis Navier, in his eminently fine design for the Pont des Invalides over the Seine in Paris, made surprisingly effective use of the construction materials to suit the requirements: cast iron for compression, wrought iron for tension; and this division is to be regarded as the starting point of more recent developments on these lines.

On the basis of the characteristics of steel construction noted in the foregoing, we must now consider how future developments may be assessed. In the first place there is the question as to what are the largest structures that can be built in the various fields of application. In the foreseeable future, bridges with spans of more than 300 m are likely to be indisputably reserved for steel construction. An investigation of the relation between length of span and weight of structure by introducing the concept of the "limiting span" which, for a given structural system and a particular material, is still just possible, and consideration of the economy within justifiable limits of application, lead to the conclusion that with our present day structural steels it is possible to build lattice-girder bridges with intermediate hinges up to a span length of 500 m (which has, in fact, already been achieved) and lattice-arch bridges up to about 600 m (for suitable soil conditions). For suspension bridges the maximum attainable span is at present probably in the region of 1,500 m. Longer spans will become possible when steels with higher strengths than those now used become available. It would appear that, in addition to ordinary structural steel St 37 and high-tensile structural steel St 52, for such large structures a steel of about the grade St 75 is perfectly within the limits of present possibilities. However, the design engineer would have to stipulate that the elongation capacity, *i.e.* the ductility, of this new structural steel must not be significantly inferior to that of the structural steels in current use, that the yield point and fatigue strength can be increased in the same proportion as the tensile strength, and that the steel can be worked and fabricated with the same means as those used for our present-day steels. This implies that its weldability must be guaranteed. With regard to steel wires of the kind used for long span suspension bridges, the question arises as to whether it would not be possible to produce a wire of increased strength and, at the same time, improved ductility. The drawn wires as now produced for suspension-bridge construction have a relatively high content of carbon, namely, about 0.8%, and carbon is in fact the cheapest way to increase the strength. It should, however, be economically possible to increase the strength by means of other added elements without entailing any greater risk of wire fractures.

In the domain of steel-framed buildings, the Empire State Building has a height which will probably not have to be exceeded for any compelling economic reasons in the near future. The cost of such a steel-framed building increases progressively with the height, and very great heights are economically justified only in places where the price of land is very high. Indeed, it can be assumed that in the case of the Empire State Building, too, adequate utilisation of the plot in economic terms would have been possible with a building of smaller overall height. In this sphere, therefore, the present resources of steel construction are likely to be able to meet the requirements of the near future.

Single-storey shed-type buildings which have to cover large areas, as far as possible without intermediate supports, can nowadays also be constructed on a scale that can fulfil any reasonable requirements. In this field the present possibilities justify the prediction that, in the foreseeable future, steel construction will continue fully to meet the requirements applied to it, especially when it is borne in mind that new structural forms, such as plate-type structures or suspended roofs, permit further development in their range of application to this class of buildings.

In the three above-mentioned fields of structural engineering — bridge construction, steel-framed buildings and shed-type buildings — steel construction has in recent years been up against keen competition from prestressed concrete. Such competition is quite necessary. It compels us to give a high qualitative performance, and the competing methods of construction may also have a stimulating effect upon each other. Some conditions must be imposed upon this competition, however : for both methods the same fundamental requirements and the same technical quality must be stipulated for the same price. In this connection quite a number of cases could be indicated where the principle of equality of rights of the two construction methods in competition has not been observed. Medium-span bridges have been built in prestressed concrete which, under equal conditions, could have been built just as efficiently and economically in steel. But the desirable equivalence of the construction methods has not been attained with regard to technical quality (and therefore the expected service life of the structures) either. A steel structure does indeed require maintenance to protect it from corrosion — the cost of which is usually exaggerated — but with proper maintenance the life of a steel bridge is limited only by a considerable increase in the traffic loads crossing it. As yet we know too little about the long-term behaviour of prestressed concrete structures. However, the problem of relaxation, *i.e.* the decline in stress that occurs in highly stressed steel element, merits attention. High-strength bolts can be periodically retightened in order to offset any relaxation phenomena that occur, whereas no retightening can be done to the cables in prestressed concrete construction, where the loss of stress is further increased by creep of the concrete. I need not go into a detailed examination of the inferences that can be drawn from this regarding the long-term behaviour of such structures. Attention must be called to one other point, however : in suspension bridges high-tensile steel wires are stressed to not more than 40% of their tensile strength, whereas in prestressed concrete construction in some countries similar wires (usually somewhat more highly drawn) are allowed to be stressed up to 70% of their tensile strength. This definitely proves that the conditions as to technical quality are not the same in the two methods of construction.

Framed buildings of reinforced concrete and prestressed concrete construction have already been constructed to heights corresponding to about a third of the height of the Empire State Building. In the competition between the steel and the prestressed concrete structural framework it should be considered that, because of the larger cross-sectional dimensions of the latter (and, more particularly, of the concrete columns in the bottom storeys), significantly more effective space is lost — for a given area on plan — than in steel-framed buildings. This difference should be correctly evaluated by a comparison of the economy of the two methods of construction.

In the construction of shed-type industrial buildings, structural steelwork has the often important advantage over concrete construction that services and equipment can more easily be fixed to the steel framework and also that structural alterations associated with a change in the purpose for which the building is used can be carried out more simply than in the case of concrete buildings.

The conditions to enable steel construction to compete successfully with its rival methods of construction are therefore entirely satisfied, provided that it is endeavoured in each individual case to produce a high-quality structure. This requires, however, that the structural system should conform as closely as possible to a natural pattern of forces and this in turn calls for realistic structural calculation. Our methods of calculation will have to be developed in this direction. The calculations constitute the basis of the structural design, and they should therefore, at all stages, reflect the actual pattern of forces in a form that can readily be visualised and checked. As already stated, analytical methods often fail to satisfy this condition, and for this reason numerical methods of calculation must also be developed. This also is intrinsically in conformity with the nature of the problem presented by a structural design calculation : it must give us the significant numerical values that characterise the pattern of forces.

In the technical literature we often find structural calculation methods which neither fulfil the requirement that they should be capable of visualisation nor are adequately representative of the actual pattern of forces nor again are suitable for practical application by the designer. Unrealistic methods call to mind the caustic assertion by the British engineer Thomas Tredgold (1788-1829) in his fine "Practical Essay on the Strength of Cast Iron and Other Metals", namely : "The stability of a building is inversely proportional to the science of the builder".

The object of the design calculations is the so-called "stress analysis", which has two functions to perform : on the one hand, it must tell us the magnitude of the stresses which will occur in the structure under working load and, on the other hand, it must ensure adequate structural safety. In normal cases (Fig. 8), in which the stresses increase proportionally with the loads, these two functions of the stress analysis coincide. However, in quite a number of structures this is not so. For example, in the case of an anchored suspension bridge, which conforms to a stress theory of second order, the stresses in the stiffening girder increase less rapidly than do the loads; hence the determining requirement is the analysis of the maximum stresses occuring under service conditions, and no verification of safety is needed. Conversely, in the case of, say, a portal frame (*e.g.* the tower of a suspension bridge) the stresses under vertical and lateral loading increase more rapidly than do the loads, and here it is essential to ensure adequate safety against, for instance, the stress reaching the yield point; on the other hand, in this case it is unnecessary to check the stresses under service conditions, since these stresses must be smaller than the normally permissible stresses.

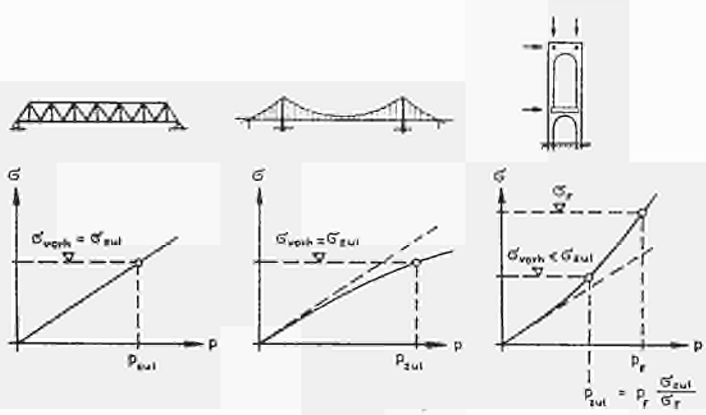

Fig. 8

A further result of the existence of this dual function of stress analysis is that the design of statically indeterminate structures by the so-called ultimate-load method is incomplete and therefore inadequate, inasmuch as such a method tells us nothing about the stresses under working-load (*i.e.* service) conditions and therefore gives no information on the essential fundamentals of the structural design. Also the assumed plastic hinges do not, in reality, develop with any precision but, at best, only very approximately.

Outstanding structures that serve as pointers in new directions of engineering development, such as O.H. Ammann's George Washington Bridge, can be created only on the basis of a harmonic synthesis of intuition, experience and knowledge. They are milestones in the process of evolution, and future generations will judge the technical skill of our time by such peak achievements. However, in addition, they are perhaps of equally great significance to ourselves : they have the general effect of widening our experience and increasing our knowledge and ability, and they thus decisively contribute towards improving the quality oı the many ordinary structures that are built too. They thus promote general development. Steel constructoin can point to a number of outstanding achievements and can therefore confidently look forward to iruitful further development of this construction method as a whole.

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[3] G. Winter: "Performance of Thin Steel Compression Flanges," 3rd I.A.B.S.E. Congress, Liège, 1948, Prelim. Report, p. 137.

[4] F. Stüssi: "Zur Bemessung von Leichtbauten aus Stahl" ("The Design of Lightweight Steel Structures"), 5th I.A.B.S.E. Congress, Portugal, 1957, Final Report.

Description of photographs

1 — Severn Bridge near Coalbrookdale 1777-1779.

2 — Suspension bridge across Menai Straits, 1818-1823.

3 — Britania Bridge, 1844-1850.

4 — Vistula Bridge near Fordon, 1890.

5 — George Washington Bridge, 1931.

6 — Brooklyn Bridge over the East River, New York, 1883.

7 — George Washington Bridge with second deck added, 1962.

8 — Narrows Bridge, New York, 1964.

9 — Road bridge over the Firth of Forth.

10 — Bronx-Whitestone Bridge, New York.

11 — Quebec Bridge (interior view).

12 — Empire State Building, New York.

13 — Eiffel Tower (first design by M. Koechlin).

14 — Palais des Expositions, Geneva.

15 — Mustermesse, Basle.

16 — Aircraft hangar at Zürich Airport.


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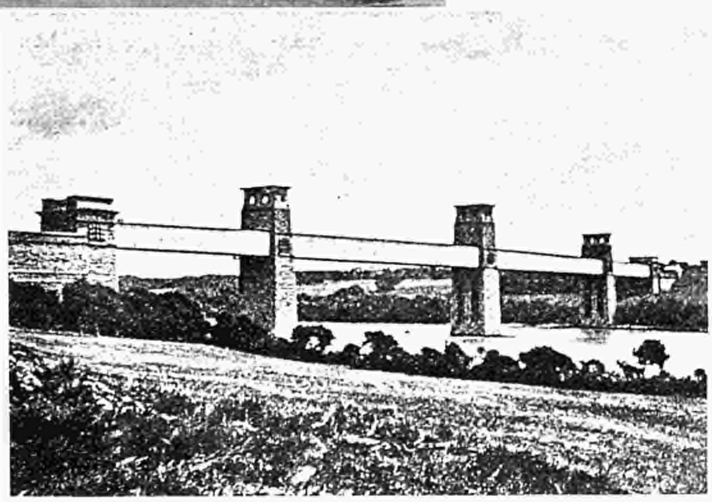
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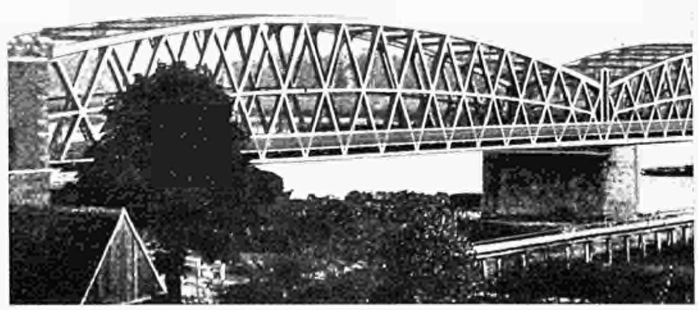
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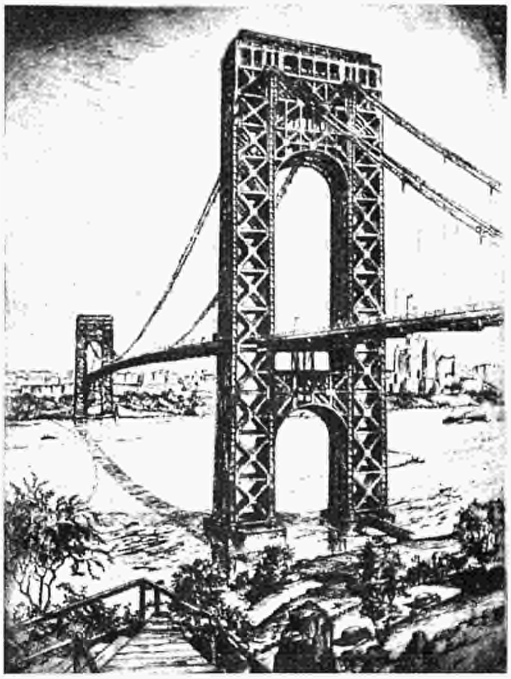

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my dear friend Prof. Dr. Fritz Stüssi
best wishes Aug 29/1962
Othmar H. Ammann

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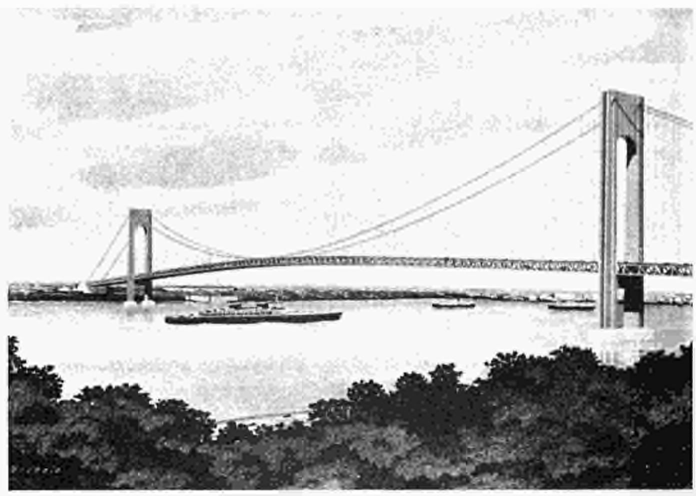

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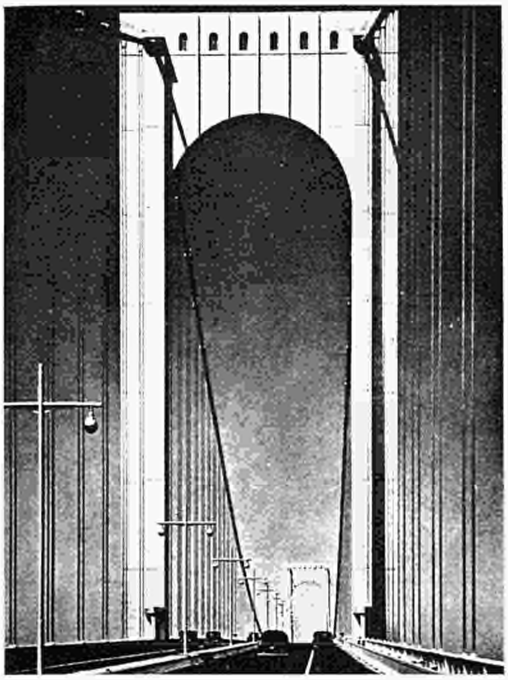
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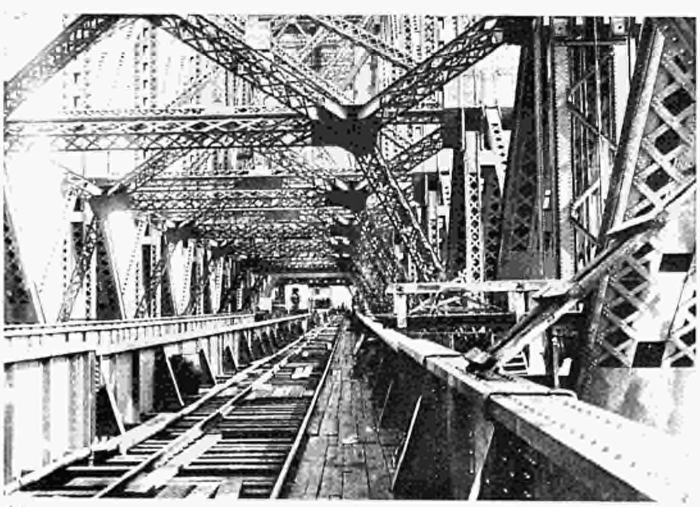
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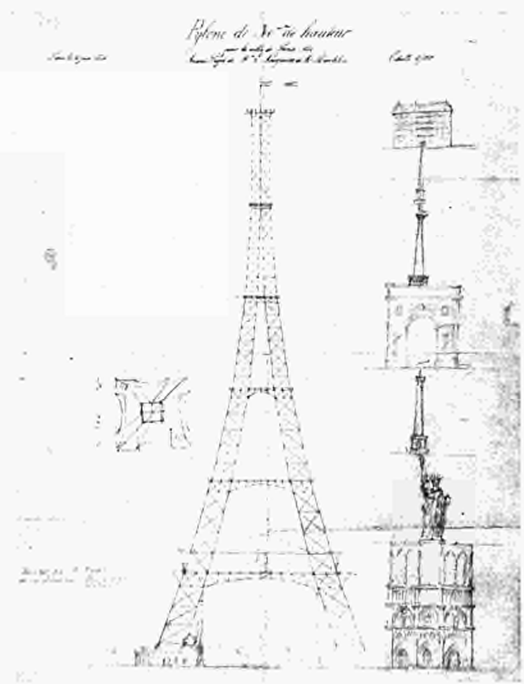
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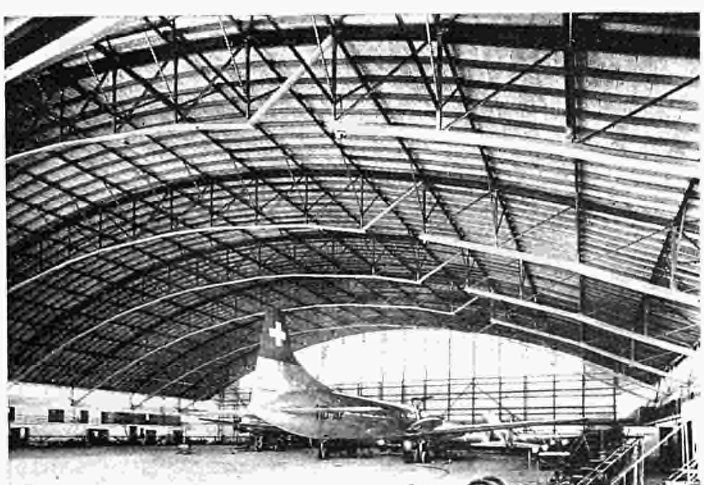


16

Improvements in the Properties of Steel used in Building and Construction

(Original text: French)

Pierre COHEUR

The subject and scope of my talk are pretty clearly indicated by its title. At first sight it may look easy enough to deal, in the time allotted to me, with the improvement of the various properties which steel offers for exploitation by intelligent builders and architects.

When looked at more closely, however, the task is not so easy after all; in fact, before long you find that a whole book would hardly suffice to do justice to it.

This is because the characteristics which distinguish steel from other materials cannot be enumerated in a few words.

While its strength properties and weldability are steel's great virtues for constructional purposes, it possesses other physical properties which are all important for certain applications, and which steelmakers have consequently been busily extolling for the past twenty years. These include its fatigue strength, resistance to corrosion and to ageing, creep strength, suitability for shaping, machining, enamelling, and so on.

In addition, with progress in technology and in particular rolling practice, the geometrical characteristics of the conventional products, such as double T beams, have been greatly improved; furthermore, the steelmakers have developed new cold-shaping or hot-extrusion processes, which enable them to produce new shapes of unlimited variety, and open up a new field for the ingenuity of architects and constructors.

The field I would have to cover is therefore very wide, and in order not to exhaust your patience, I propose to survey only a part of it by following the evolution of two constructional characteristics which are specific to steel : its mechanical strength and its resistance to brittle fracture.

Mechanical strength

As a strengthening element in constructions, steel is unquestionably a relatively recent, modern material. It is less than a century since acid-Bessemer, basic-Bessemer and open-hearth steel first showed that it was possible to construct framework in which the loads and principal stresses were supported by elements resistant to tension, bending and torsion.

Before then, civil engineering consisted essentially in utilising the resistance of materials to compression stresses, and all the designs of architects, from pyramids to cathedrals, were based on this fundamental principle.

I shall illustrate this point first by referring to the fourth century Iron Pillar near the Outb Minar, which is one of the sights of Delhi even today for students of archeology and those having to do with iron and steel (1).

Next I refer to the first steel bridge ever built. This was a single span erected in the eighteenth century by the iron smelter Darby (1779) over the Severn near Brosely (2). In accordance with the principle of the traditional architecture of the period, this structure, with its cast-iron arches and rings imitating the arches and arch stones of a masonry bridge, was certainly not an example in which the metal was used for its specific properties; it would be difficult to detect in it any elements subjected to tension or bending.

If to some this pioneer structure stands for lighter weight building and prefabrication in metal, to the steelmaker it is also the starting point of a new development in the iron and steel industry : its builders, the Coalbrooke Iron Works, utilised their experience to construct many other bridges which they supplied to all parts of the world.

At the beginning of the nineteenth century (1823), progress-minded constructors forsook cast iron and turned their attention to the mechanical characteristics offered by wrought iron; as representative of their work I have chosen the Eiffel Tower (1889), the boldness of conception of which brings out well the potentialities of steel as a building material.

It is quite natural that following the development of manufacturing processes, constructors became interested in steel, despite its higher cost and the criticisms voiced by specialists as to the homogeneity of its chemical, mechanical and structural properties.

Of the first major steel constructions, I consider the Forth railway bridge (1883-1890) one of the finest examples (3). Impressive in its size and the originality of its structure, this work foreshadowed the modern method of construction in the tubular elements forming part of it. In selecting, as long ago as 1883, mild steel instead of wrought iron for its construction, the builders courageously took up a position in the sometimes bitter controversy between the champions of the two materials.

The mild steel generally used at that period had the mechanical properties shown diagrammatically in figure 1, and which may be regarded as the starting point of the properties of steels.

The different rectangles represent successively from left to right :

— tensile strength;

— elastic limit;

— ductility, measured by elongation on rupture under tension.

Actually, I ought to have omitted the elastic limit because at that time it was not used in calculations and was rarely measured.

For nearly fifty years, architects and constructional engineers used this steel in their structures, making use of the well-proven construction and assembly methods to which they were accustomed.

The steelmaker for his part improved the quality of mild steel, which even to-day is still the basis of the production of steels intended for use in steel structures.

Apart from some very costly attempts carried out in nickel-chromium steel, it was not until the years 1925-1930 that the first important industrial productions of steels specially made to meet high working stresses, made their appearance in steel frameworks.

The chemical compositions and mechanical properties were rather variable, but the formula of a first manganese-carbon steel was seen to be gradually emerging.

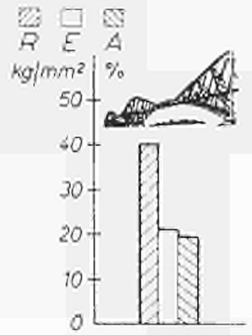


Fig. 1

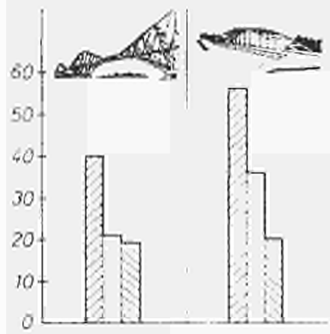


Fig. 2

As St 52, this steel was developed mainly in Germany, thanks to outstanding co-operation between constructors and steelmakers.

Figure 2 summarizes in the same form as before the substantial progress made at that time.

It should be emphasized, however, that these were pioneer ventures, and while a few Vierendeel or bowstring bridges were built of St 52, such applications were in the nature of prototypes and practically no standard specifications provided for its use.

Taking this steel as a prototype, then, we can see what enormous strides have been made in regard to high-strength steels in the last twenty years.

As an indication of the background I may recall first of all that as the result of political and economic circumstances, steelworks have been having for the last quarter of a century to improve or renew their production equipment. Thus, new steelworks have been built, using enormous quantities of oxygen and offering a wider range of steelmaking.

Furthermore, the possibilities of cold and hot shaping of steel have been considerably increased, not only by the erection of bigger rolling stands or mills, but also by the use of new rolling methods.

Finally, by a strict control exercised in all stages of manufactures, accidental variation of the mechanical properties of the steel have been reduced, together with the dimensional tolerances.

Furthermore, spurred by competition, the steelmakers are utilising all the resources of metallurgical science, in particular for producing economical steels combining high strength, ductility and weldability in proportions satisfying the requirements of modern constructions.

Indeed, these requirements are becoming increasingly complex with the improvements in the forms of the structures. It may be said that in the construction of conventional frameworks, the time is approaching when the materials to be employed ought to have physical properties adapted to the function of the elements of the structure for which they are intended.

The grades of steel cannot therefore easily be enumerated without taking into account all the subdivisions which emphasize the particular properties of resistance to ageing, fatigue strength or corrosion resistance, suitability for shaping, machinability, etc.

However, to confine ourselves to the general tendencies dominating the evolution of the properties of constructional steels, it seems to me that the current development in the use of high working stresses, while maintaining the same level of ductility, may be summed up in the four solutions illustrated in figure 3.

In the first place, it is a fact that a first steel of high elastic limit has finally won a place as meeting an almost international standard.

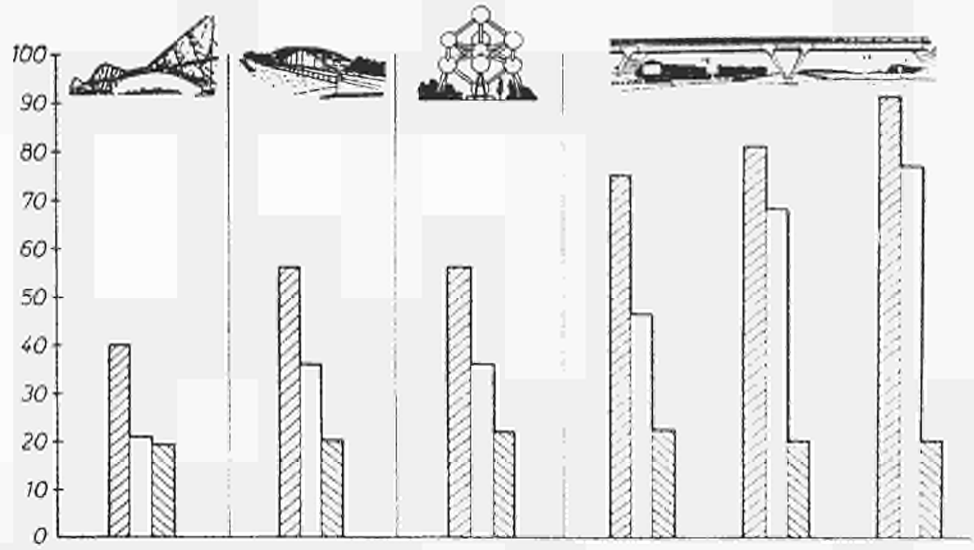


Fig. 3

This is the grade denoted, according to the E.C.S.C. symbols, by the designation Fe 52, the elastic limit of which is 22 to 23 tons/sq. in. (34 to 36 kg/mm²) according to the thickness, and the tensile strength from 33 to 40 tons/sq. in. (52 to 62 kg/mm²). This European steel corresponding to the pre-war prototype St 52 mentioned earlier, is now standardised in every country from Japan to Russia, via the U.S.A.

Generally made without any alloying element apart from Mn and Si, this carbon steel owes its service properties to the qualities of a fine-grain ferritic structure.

If the means of production permit, this steel has good guarantees of regularity, and its ductility and weldability properties are very close to those of mild steel.

To illustrate industrial progress, it is always easier and more striking to show the performances obtained, but these are not always the performances most useful for economic development; constancy and guarantee of quality as improvement in the technical manufacturing balance-sheet are often more effective arguments.

This is the case for steel Fe 52 of the existing standards; its properties, shown in Fig. 6, do not differ much from those of before the war, but these values, which were then performances, are to-day current standards, guaranteed by statistical manufacturing controls. In numerous civil engineering constructions, such as bridges and other structures, this first steel of high elastic limit has been found to be very economical, and we often hear of large structures working out over 20% lighter than if they had been made of mild steel.

In shipbuilding or the construction of electric pylons, in mining equipment or rolling stock, the field of application of this steel will certainly be extended still further. Perhaps tomorrow we shall see it used in the framework of buildings, as a permanent substitute for the current grades Fe 37 and Fe 42, as long as, of course, the elastic strains remain permissible, and the projection of the constituent parts does not give rise to buckling. I make this reservation because the modulus of elasticity of every type of steel depends on the atomic structure of the iron and is, therefore, a physical constant.

As a representative application of steel Fe 52, I have taken the construction of the Atomium at the 1958 Brussels International Exhibition (although this was by no means the first construction in this steel, and numerous examples may be quoted in European countries and in particular in Belgium). I think, however, that this is a construction with which all of you will be acquainted.

This steel Fe 52 is only a first stage towards high strengths. The steelmaker in fact has three principal means available for increasing the strength value, *i.e.*:

— strain;

— addition of alloying elements;

— heat treatments.

The first of these is employed in certain processes, and there are many cases in which strain hardening is utilised for obtaining high strengths. Take for example wires for making cables, the tensile strength of which may attain 127 tons/sq.in. (200 kg/mm^2); this, however, is a process which cannot be generally employed owing to its effect on the other properties of the metal, especially its ductility and weldability, so that its application in steel constructional work will be limited.

On the other hand, additives and heat-treatments, employed either separately or usually in combination, have made it possible to produce sheets and sections of any gauge, the strength properties of which are superior to those of steel Fe 52, while retaining a high ductility value.

Three examples selected from many others are shown in the three right-hand columns of figure 3.

The first relates to low-alloy steels supplied in normalised condition. The improvement in the strength properties results more particularly from the specific action of the additives on the ferrite and pearlite. For this type of steel the elasric limit is generally 29 to 32 tons/sq.in. (45 to 50 kg/mm^2) and the tensile strength from 38 to 48 tons/sq.in. (60 to 75 kg/mm^2).

The other two examples relate to the second type of steel, that is to say, heat-treated low-alloy steels. In this case, the added elements have a threefold function : first, to displace the austenite transformation curves and thus help to produce metastable, as quenched structures, subsequently stabilised by tempering; second, to improve the intrinsic properties of the structural constituents; and third, to produce a very fine hardening precipitation. These steels are supplied and used in the treated condition, and their elastic limit lies in the range from 42 to 48 tons/sq.in. (65 to 75 kg/mm^2). Owing to their composition or the state of their microstructure, such steels, whether of the first or second type, no doubt require certain precautions in their use for shaping or welding, and perhaps also certain adaptations in the actual design of constructions.

These are criticisms which are frequently made and merit detailed examination by the designer before any practical work is undertaken.

These criticisms, however, are of the same order as those of former times when steel took the place of wrought iron or when the designer developed welding; I do not think the use of these materials in steel constructions will be retarded by such criticisms.

The practical problems involved are merely a reason for design engineers and constructors to make a more thorough study of metallurgical physics and an argument for closer co-operation between producer and user.

To sum up, where the elastic limit of the steels currently employed used to be 15 1/4 tons/sq.in. (24 kg/mm^2), it is now 23 tons/sq.in. (36 kg/mm^2) and will rise as time goes on to 28 1/2 to 32 tons/sq.in. (45 to 50 kg/mm^2).

This advance as regards strength will, however, be of little practical value unless immediately followed by corresponding progress by the architect, design engineer and constructor, whose plans and methods must take the fullest advantage of these new properties the moment they become available.

I realize that this problem of utilisation is complicated and cannot be reduced to the simple arithmetic rule of increasing the work load in proportion to the increase in the elastic limit. Nor do I wish to enter a technical field in which I would very soon find myself out of my depth, but I must state that by taking into account the modulus of elasticity, choice of sections, reduction in dead weight, modification in the assemblage joints and various other constructional factors, by resorting to new methods of calculation, the permissible stresses have successfully been increased to a quite unexpected extent.

Thus, for comparable loads, the permissible stresses in welded frames, which were 10 1/4 and 11 1/2 tons/sq. in. (16 and 18 kg/mm^2), respectively, for steels F 37 and F 42, have been increased to 15 1/4 tons/sq.in. (24 kg/mm^2) for steels of type F 52 and to 19 tons/sq.in. (30 kg/mm^2) for steels with an elastic limit of 28 1/2 tons/sq.in. (45 kg/mm^2) (Fig. 4).

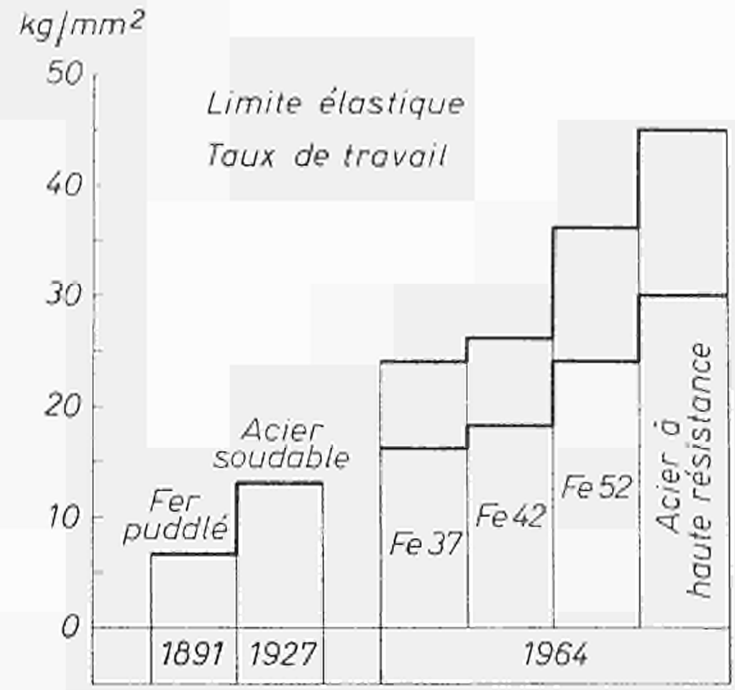


Fig. 4

When we think that a French Ministerial Circular of 1927, superseding the previous official regulations of 1891 and 1915, daringly fixed the maximum possible tensile and compressive stresses at 8 1/4 tons/sq.in. (13 kg/mm^2), it is obvious that we have come quite a long way since then.

This we owe principally to the constructors of rolling stock and hoisting and handling equipment, as well as to the builders of large bridges, who where the first to be faced with the problems of weight reduction and high stresses.

It may be regretted that not all constructors of steel frameworks have given the same attention of these points.

In Europe, frameworks of buildings of high-strength steel are exceptional, and there is still a widely held opinion that these materials do not offer any real advantages in such structures.

This opinion should certainly be reconsidered, and I trust that our technical working parties will discuss this in the next few days.

Have we not in fact been informed by a recent American study that the Empire State Buildings could be increased in height by 13 floors if its framework had been made of steel having a high elastic limit?

Such is the general trend of the development of the strength properties of steels intended for metal constructions.

As a matter of fact, the technical possibilities of the steelmaker have not been exhausted in this direction, and we are still far from the 190 to 255 tons/sq.in. (300 to 400 kg/mm^2) which are attained in certain steels.

In the case with which we are concerned, however, the problem of increasing the strength of steels is primarily an economic one, involving the following factors :

— cost of the steel in relation to unit strength;
— strength of the steel in relation to unit weight;
— adaptation of the structures to the properties of the new steels;
— difficulty of employing the steel according to its strength;
— and, finally, the regulations in force in steel construction work.

The economists will say that it is upon success in striking a balance among these various considerations that future progress in the elastic limit of steels will ultimately depend.

Allow me to say that I do not altogether believe this.

In this field, as in many others, technical progress almost always necessitates making a first step, accompanied by a temporary economic sacrifice.

If the constructor tells me that steel of high elastic limit is not used because it is too dear, the steelmaker will reply that steel of high elastic limit is too dear because it is not used enough.

To break this vicious circle, it is necessary to believe in a long-term profit and certainly show some boldness.

In Belgium, as elsewhere, steel Fe 52, nowadays a conventional and economical material, first gained a real footing purely because some constructors and public authorities had the courage to bank on a particular reading of the future.

I am convinced that for the development of steels of higher strength, we shall still find pioneer steelmakers and constructors who are prepared to forgo the easy way and their own immediate advantage for the sake of experimentation, but I think that this progress could be accelerated considerably if organisations such as ECSC could encourage the realisation of such experiments in the general framework of a steel promotion policy.

Weldability - Brittle fracture

Apart from strength and perhaps ductility, the most specific constructional property of steels is their weldability. Very largely used in steel constructions as a means of assembly and for the fabrication of composite elements, the possibility of assembling by welding is certainly the strongest argument for steel construction.

This advantage, without which steel would have forfeited any chance of being used in civil engineering, but with which it has most promising prospects in future constructions where prefabrication will be the rule, does, however, acquire the metal to be of the very highest quality.

Taken in its most general meaning, "weldability" denotes the whole aptitude of a steel to be employed in a welded construction.

This aptitude is rather difficult to impart to steels, because it requires the steelmaker to strike a balance between sometimes contradictory demands, and furthermore it depends in a very large measure on factors which are beyond the control of the steelmaker, such as the factors involved in the employment of the steel, the influence of the geometry of the structures, and the service conditions.

As far as the steelmaker is concerned, as you are aware, the zone of metal adjacent to the weld in particular is subjected to extremely high thermal stresses, since one of its edges is suddenly heated to the melting point of steel, while the other a few millimetres away remains at the surrounding temperature.

The conditions so produced are particularly favourable to the operation of all the factors governing the hardenability of the steel, precipitation of various constituents and atomic diffusion, so that some embrittlement of the steel is liable to result.

To reduce or eliminate this effect, the steelmaker must first of all regulate the composition of the metal, by restricting more particularly the content of carbon and hardening elements, that is, in fact, the content of alloying elements, although these are at the same time necessary for increasing the elastic limit. He is thus obliged to strike the balance I spoke of, which he can do by recourse, more particularly, to the equivalent carbon. Furthermore, to get rid of the germs of temper brittleness ("Krupp's disease") or equivalent defect, he has to eliminate as much as possible the elements which are liable to embrittling precipitation, or at least bind them in a stable complex.

The constructor, on his part, can reduce the severity of the stresses set up by welding by operating under the most favourable conditions. One of these is to reduce the temperature gradient by preheating the steel. He can also suppress the harmful effects of the thermal stresses, and in particular destroy the metastable compounds which have been formed, by carrying out treatments after welding.

This may be done on the site, but it is obvious that the working conditions will be far more favourable and their effect more certain if they are carried out in the workshop, and this is certainly a point in favour of the prefabrication of structures.

Apart from the embrittling phenomena in the vicinity of the weld, it is also necessary to bear in mind the structural brittleness which I mentioned earlier. As you are aware, the combination of these various factors was behind the series of terrible accidents which occurred in the years 1935-1943 not only on the steel framework, but also on the shipbuilding and boilermaking sides. Steel embrittlement is perhaps the physical phenomenon which has been most studied in the past twenty years. These researches, which were costly and long, have now culminated in new knowledge and in practical results which are permitting the problem to be overcome. This new knowledge includes a matter which lies in the scope of the steelmaker and on which I should like to dwell a little : the resistance of steels to brittle fracture.

Like all metals of body-centred cubic structure, the fracture of an iron crystal subjected to simple tension generally takes place along one or other of its 48 possible slip directions. Elongation is relatively considerable and the appearance of the fracture is fibrous (Fig. 5).

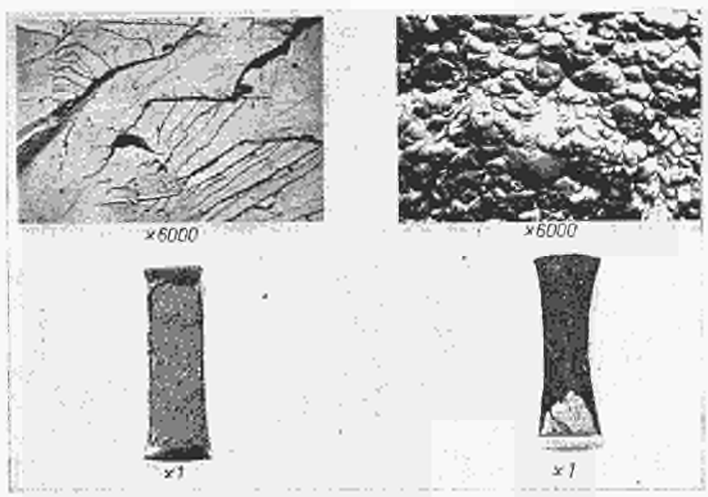


Fig. 5

If the possibilities of slip of the crystal are prevented by polyaxial stresses, or if they are frozen by reducing the temperature sufficiently, or again if the crystal is not allowed sufficient time to recover by subjecting it to rapid stress, fracture then takes place by another process, that of decohesion. In this case, the elongations are very slight, the fracture has a crystalline appearance and the metal behaves like a brittle substance.

This phenomenon may be demonstrated by subjecting notched specimens to impact tests carried out at decreasing temperatures. The results are plotted in a diagram similar to that of figure 6, in which the impact strength of the metal, that is to say its toughness, is plotted as ordinate and the temperature as abscissa.

This diagram shows that at a certain temperature, called the "transition" temperature, fracture of the metal passes from the form by simple or multiple slip to that by decohesion.

Actually (Fig. 7) this passage takes place in a temperature range, and for reasons of simplification, it has become customary to consider the transition temperature corresponding to a certain fracturing energy "N".

This transition temperature naturally depends particularly on the shape of the test piece, the type of the impact test and the energy level selected. It is, therefore, a conventional temperature permitting steels to be classified according to their resistance to brittle fracture in a relative order of merit.

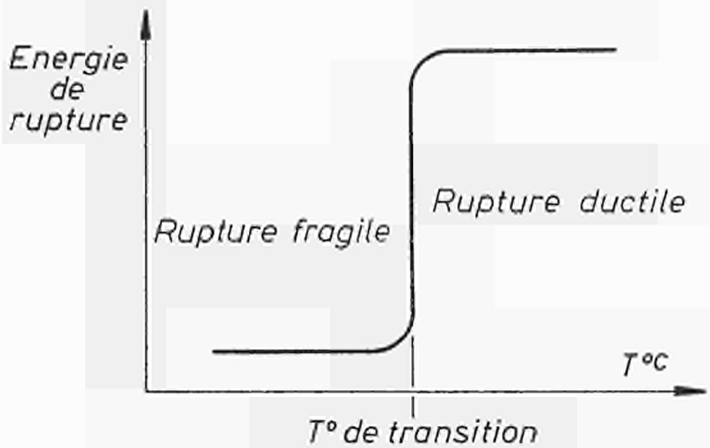


Fig. 6

By thus basing itself on this qualification criterion, metallurgical research has made it possible first to analyse and then to control the elements of chemical composition and microstructure which are favourable to it.

The question of the brittle fracture of steel assemblies or constructions is certainly not yet exhausted and there still remain to be solved important problems of correlation between the conventional transition temperature of steel and the service behaviour of structures in which the stressed state and rate of application of the stresses play a considerable part in the behaviour of the metal.

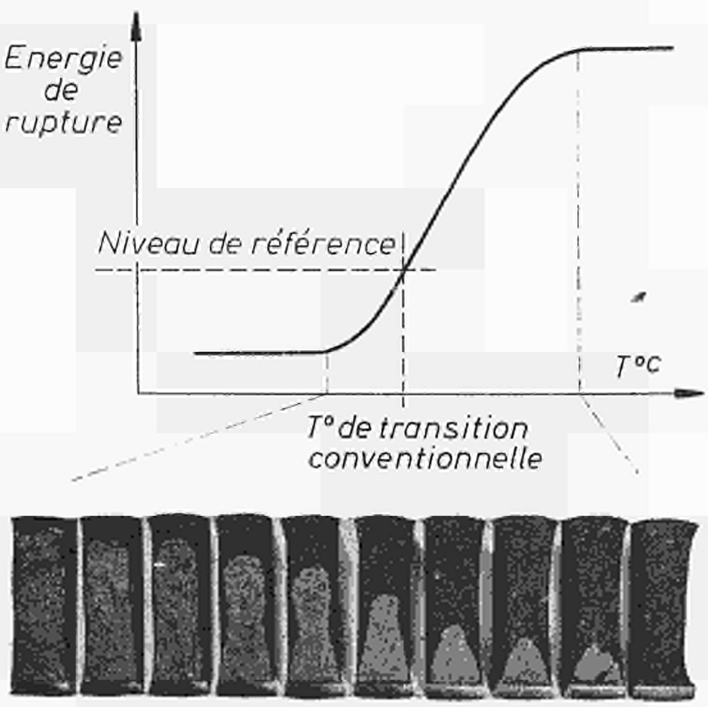


Fig. 7

The important point so far established is that the quality of resistance to brittle fracture, as defined by a transition temperature, may be used for defining "classes of steel on a weldability scale" in the same way that the strength properties — tensile strength and elastic limit — have served for the classification of steels in "grades". This transition temperature, therefore, must not be confused with that of the assemblies, still less with the service temperatures of the corresponding constructions.

I shall illustrate the classification just mentioned first of all by means of the three curves of figure 8.

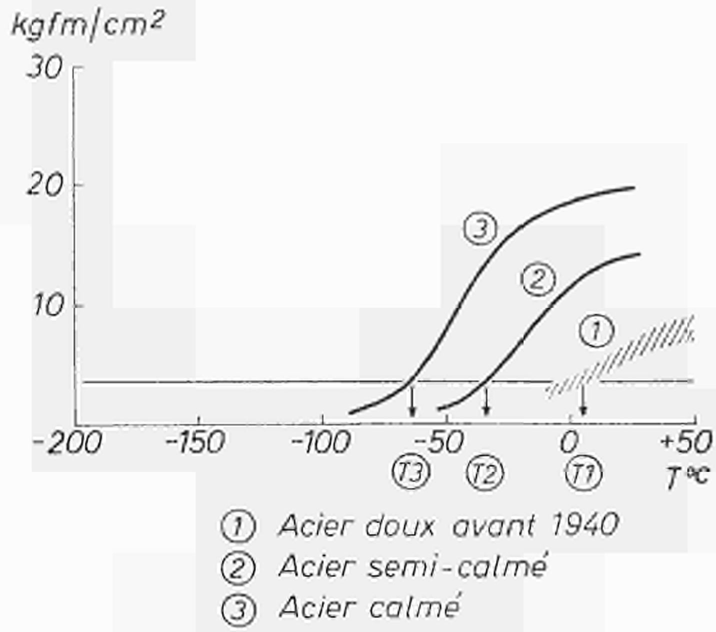


Fig. 8

The curve on the right relates to the rimming mild steels of the 1940's. Such steels are undeniably weldable and *a priori* offer guarantees of resistance to brittle fracture quite sufficient for most of the elements forming welded frameworks.

The middle curve represents a typical case of killed or semi-killed silicon steels, and the left-hand curve of aluminium killed fine-grain steels.

For a welded steel framework constructed in our climate for static stresses, it may be asked whether these levels, which we elliptically term "weldability" are not amply sufficient in themselves.

As regards the service behaviour of present-day constructions, this is a fair enough question, but nevertheless there are special circumstances where fuller guarantees are necessary; this is particularly so in the case of products of considerable thickness or products that, in service, have to undergo plastic deformation and thermal ageing, which are known to have embrittling effects, or again in the case of mobile constructions intended to support dynamic loads or operate at very low temperatures.

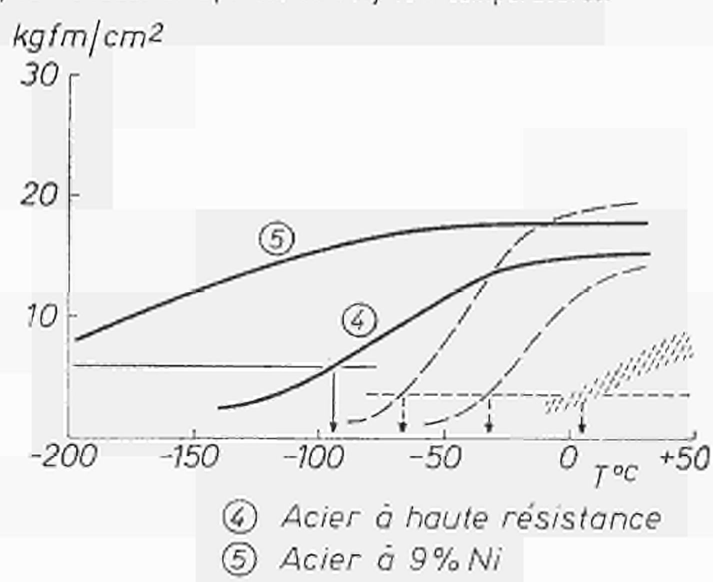


Fig. 9

For these applications special steels, generally high-strength steels, have been perfected in the course of recent years.

The transition temperatures obtained are quite surprising. By added elements or by heat treatments, it has been possible to reduce the transition temperatures to — 100° C or — 200° C (Fig. 9).

Two typical examples are shown in this figure. The first relates to a low-alloy steel quenched and tempered, and the second to an alloy steel containing among other elements 9% nickel. It may be noted in passing that maximum toughness is not essential; what is important is that transition from ductility should occur only at low temperatures.

Owing to their appreciably higher cost and the stricter conditions of use necessary, these steels are obviously intended only for special applications. In mentioning them here, my intention above all is to show that the steelmakers have now mastered the problem of ensuring resistance to brittle fracture of steel — one of the finest examples of the effectiveness of basic research in physical metallurgy.

It is true that it would be advisable to prosecute the study of the problem by measuring the transition temperatures of welded assembly joints. This technique is definitely calculated to get closer to practical requirements.

The quality of high weldability being a costly requirement for the steelmaker, even in the case of plain carbon steels, economy of construction requires the constructor to select the weldability class with discernment in terms of the risks of the construction, such as the stresses on the various elements and the form in which the latter are employed.

In this field also, much progress has been made, and credit is due to the technical committees of E.C.S.C. whose hard and unenviable labours on standardisation have gradually succeeded in classifying steel products with a view to greater economy.

It is too often forgotten that true industrial progress is not always spectacular, and that the openings for improved performance have sometimes to be sought in practical operational details or the choice of products.

Conclusions

This brings me to the end of my remarks; it remains for me to sum up.

Perhaps you will be surprised that in dealing with the evolution of the intrinsic properties of steel, I have not emphasized, or even mentioned, the development of steelmaking processes, which is certainly the European steel industry's pride. In the field of steel construction and for mass products, the various processes may each supply the quality of steel which the customers demand, so that the choice of one process or another depends on economic, local or raw-material-supply conditions and is therefore a matter for the steelmaker.

Furthermore, it is a well-known fact that quality is today largely governed by the ladle treatment, rolling and post-rolling processing of the metal.

And on this subject, I think I have demonstrated the efforts which the steel industry has made to meet consumer's wishes. (4)

These efforts, however, will only be fully effective if the consumer is able to make the very most of all the possibilities offered to him, and as I have told you, this postulates in the first place a co-operative drive by architects, design engineers, constructors and steelmakers to standardize the constituent elements of steel construction work, and to ensure the maximum of prefabrication in the workshop and the minimum of assembly and finishing work on the site.

Increased utilisation of steel also implies that the public authorities and services should adjust their regulations to keep pace with technical progress.

A Congress like this, which is the first of its kind, undoubtedly establishes the soundest basis for this co-operation, and the High Authority is to be congratulated on its intelligent action in organizing this occasion which will, I feel certain, result in further advances in constructional steelwork. To me it is a good omen that the High Authority should have chosen the city of Luxembourg as the venue of this Congress.

For Luxembourg has the unusual privilege of possessing both a stone bridge, the Pont Adolphe, whose span of 275 ft. (84 m.) is for stone an almost unequalled achievement, and a bridge of steel plate, the Pont Grande-Duchesse Charlotte, now under construction, the modern lines and structural lightness of which will doubtless remain a pattern of its kind.

This contrast and parallelism between two achievements and two periods brings us right to the heart of the matter, showing as it does just what architects and constructors, in co-operation with the steelmakers, can produce in such splendid material as steel.

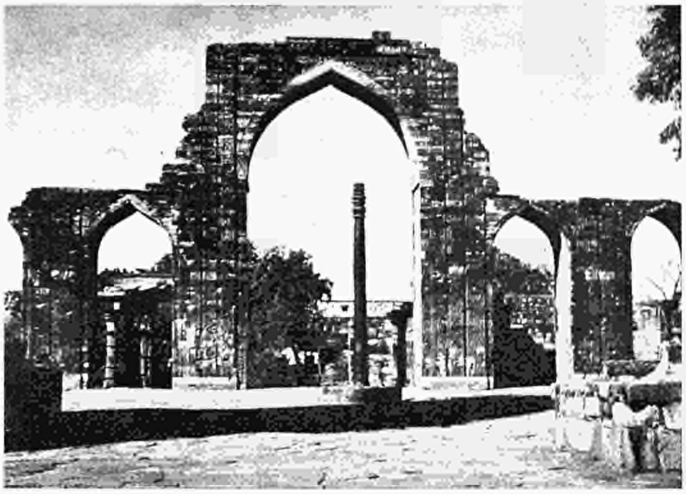
Description of photographs

1 — Fourth-century Iron Pillar near the Outb Minar.

2 — Severn Bridge near Brosely.

3 — Forth Railway Bridge (1883-1890).

4 — Europe Bridge, Austria.



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4

Official Regulations concerning Steel Constructions in the Community Countries

(Original text: Italian

Letterio F. DONATO

The subject of Regulations on Structural Steel Construction in the Countries of the European Common Market of which the High Authority has honoured me by its invitation, could have been dealt with by research workers and specialists far better qualified than myself in European matters. I have agreed to deal with the subject at this Meeting, not out of any presumption, but as a dutiful acknowledgement to my Colleagues who have shared with me the responsibility of drafting the Italian Regulations issued in July, 1963, and at present being applied experimentally. The Commission appointed for that task had from necessity, in the course of its work, to make a diligent comparative examination of similar foreign Regulations. Therefore it seems to me that in making known the results of their recent activity it might in some measure contribute to the common effort of reaching in the not too distant future some European Regulations.

In order to keep my exposition within reasonable limits, I hope that I shall be allowed to confine myself to the essential matters even if I have to leave out others equally important. This is because in order to justify some aspects of the Italian Regulations, I shall have to refer occasionally although briefly, to the experience of a wider international field.

I trust also that I shall be forgiven if, conversely, my exposition does not refer directly to the Luxembourg and Dutch Regulations, which undoubtedly would have led us to interesting comparisons. My own experience of these regulations however is too limited to enable me in the time available to deal fully with the topic as fully as I would have wished.

Constructional steels

The examination of the grades of steel used by the varoius countries in structural steel constructions, in view of a possible international standardization, has been the object of investigations by the Second Commission of the European Convention of the Structural Steel Construction Associations and also by a special working group of the E.C.S.C. The problem was rather difficult owing to the large variety of steel products involved, not always having well defined properties, which are currently used. To give a brief idea, a few examples may suffice.

The Belgian NBN 1, in Art. 305.1. of Table IV, chapter II, envisages three varieties of A 37, three of A 42, two of A 52 and one of A 00; the third paragraph of the same article also covers the use of any other steel which may be considered suitable for the purpose.

The DIN 17100 lists a wide range of products, but Structural Steel Constructions use essentially type St 37, in the varieties 37, 37.2, 37.3, and type St 52.3.

The French CM 56 Regulations envisage two qualities, ADx Charpente 35/46 and ADx 35/50. For the conventional limit of 0,2, now not guaranteed, a tensile strength is envisaged as a provisional measure of 24 kg/sq.mm., with a reduction of up to 20 kg/sq.mm. for thicknesses exceeding 20 mm. It is permissible to use steels having higher properties, in any case with ultimate elongation not exceeding 18%, provided the application is laid down in special specifications.

BS 449 : 1959 specifies BS 15 as a steel for current use and BS 548 and BS 968 (for welded constructions) as high-strength steels. It is advisable to replace BS 15, also generally suitable for welding, in the case of thicknesses exceeding one inch or when there is a risk of brittle fractures.

The Italian CNR-UNI 10011/1963 envisages the employment of two types of steel : type 1, also called standard, and type 2, high-strength. The former comprises A 37/42 and A 42/50, both with a yield point of 24 kg/sq.mm., ultimate elongation on a long bar not lower than 25% for A 37 and 23% for A 42; the latter A 52/62 with a yield point of 36 kg/sq.mm. and ultimate elongation, still on a long bar, not less than 22%. For both types, the Charpy V resilience must not go below 3.5 kg/sq.cm. at 20° C. It is also permissible to use, in either of the above types, all steels belonging to the Italian Standards in respect of which, in addition to the requirements prescribed by the Italian Standards, the minimum properties laid down by CNR-UNI 10011 are guaranteed. Lastly, it is possible to use non-standard steels, provided they meet the requirements laid down in the Regulations.

These summary indications, although not altogether discouraging to the definite prospect of placing at the disposal of the European constructors in the near future a small number of steel products having well-defined properties. They also offer in the best possible way the requirements of structural steel work, and in any case are sufficiently representative of the considerable difficulties of the problem. Among other things, the Standard Regulations of the various countries do not always follow uniform principles in defining the materials and when they refer to similar features, these are not clear enough to identify a given material in ordre to make exact comparisons. As a preliminary, it will be necessary to give a complete list of the European structural steels so that their properties may be subsequently analysed in relation to the possible applications in the various branches of the structural steel construction.

As far as I know, this is just the method of work adopted by the European Convention in this field. Considerable progress towards standardization appears to have been recently made by the E.C.S.C.

Types of loads

NBN 1f 1959 classifies as follows, the stresses to be taken into consideration : for permanent load, extra loads, dynamic factors, snow, wind, extra loads due to the testing of lifts, goods elevators, hoisting equipment, maintenance equipment, etc. The evaluation of the wind influence is laid down in NBN 460. The extra load due to snow on the horizontal surface, constant and equal to 35 kg/sq.m. from sea-level up to an altitude of 100 m., increases until it reaches 65 kg/sq.m. at the height of 700 m. It is not taken into account when the pitch gradient is equal to, or higher than 50°, excepting where there is presumed to be an accumulation of snow the presence of which must be considered simultaneous with the influence of the wind. The temperature range is generally envisaged to be between — 20 and + 40° C., the average temperature being 10° C. A temperature differential of 10° C. must be taken into account between the parts exposed to the sun and those protected, when calculating the hyperstatic systems, especially for continuous beams.

The said NBN 1f 1959 envisages the following cases of stress :

(1) superimposition, of the most unfavourable nature, of all cases of stress including temperature variations and snow in the absence of wind ;

(2) superimposition of normal wind to the most unfavourable influence of all cases of stress, as in the previous one, excluding snow, barring special cases;

3) superimposition of exceptional wind to the most unfavourable influence of all cases of stress as in the first one, barring snow.

The Règles C.M. 1956, as in the case of other regulations, refers to special regulations or specifications, the determination of operational extra-loads, possible dynamic influences and climatic loading, such as snow and wind. In general, the standardized extra-loads are more restricted than in other countries. The coefficient of expansion of steel is given as 11×10^{-6}; heat stresses are to be evaluated for a uniform differential of $\pm$ 27° C ; possible temperature differences between one component and another in the same construction are to be taken into account.

Under N° 3.1., the Regulations envisage the following cases of stresses :

(1) Permanent load, operational or testing extra-loads, also influences due to methods of construction and assembly, temperature influence, and snow *in the absence of wind* ;

(2) As above, plus wind loading;

(3) Corresponding to the so-called *complementary verification* of safety conditions: *i.e.* permanent load calculated in the strictest possible manner and the worst conditions of all extra-loads — without snow, but increased by 25% together with the wind increased by 50%.

DIN 1050 subdivides the stress influences as follows :

— *Main loads* : permanent load, extra-loads (snow included, but wind excluded);

— *Dynamic loading;*

— *Complementary loads* : wind, braking stresses, horizontal stresses (transmitted, for example, by overhead cranes), influence of constructional or maintenance equipment, temperature influences (operational and climatic).

Special regulations, for example, DIN 1055/4 for wind, and 1055/5 for snow, lay down the methods for evaluating single loads.

The said DIN 1050 envisages the two following types of load :

— *Condition H* : worst condition of the main influences;

— *Condition HZ* : worst condition of the main and complementary influences.

The minimum snow load on the horizontal surface is given as 75 kg/sq.m. The simultaneous influence of wind and snow is excluded for a pitch with an incline exceeding 45° excepting cases of possible accumulation of snow or special circumstances calling for different rules (DIN 1055/4, No. 3).

When on a structural unit, in addition to its own weight, there act complementary loads exclusively, these are to be considered as main loads.

The Swiss Regulations of S.I.A. N°. 161-1956 also list two classes of stress influences :

— P : *main loads:* own weight, permanent or mobile extra loads, snow, centrifugal force, dynamic influences;

— C : *complementary loads* : wind, braking, starting and hoisting stresses and similar influences.

The checks must be made for the worst combinations of both the loads P only and the combination of loads T = P + C, unless one of the two conditions of load is undoubtedly preponderant.

The Italian Rule is contained in CNR-UNI 10012 which a few months ago entered the experimental application stage. This envisages the following types of load :

— *Condition I*, which embraces in the most unfavourable manner the *main influences:* permanent load, operational extra-loads, snow, earth pressure, dynamic influences;

— *Condition II*, which — in addition to the influences of the former — embraces in the most unfavourable manner the *complementary influences:* wind, temperature variations, contraction, fixing imperfections, assembly faults.

The complementary influences are to be considered as main ones when, in association with the permanent load they create a preponderant influence in relation to that of Condition I.

Fig. 1

The minimum extra-load of snow on horizontal surfaces is calculated differently for Zone I (North-East of the Peninsula) and Zone II (South-West and Islands) of the National Territory, as shown in a special map of Italy (Fig. 1)

— Zone II :	60 kg/sq.m.
— Zone I :	
for heights up to 300 m.	90 kg/sq.m.
for heights h (in metres) higher	90 + 0.15 (h — 300) kg/sq.m.

On a similar basis to that used by DIN 1055/5, as the gradient of the pitch rises, starting from 20°, the load calculated as above can be reduced by 2 1/2% for each degree of rise above 60°, when it is reasonable to assume that the snow would not remain on the roof and can be neglected. Snow pockets in V-jointed surfaces are deemed to exert a load with a specific gravity between 250 and 500 kg/sq.m., according to the degree of consistancy which may be presumed.

The Rules relating to the evaluation of wind pressure are very detailed. This subject kept the Research Commission very busy working on the data of a large experimental documentation in order ro determine the kinetic pressure exerted by the wind.

For heights above the ground of up to 20 m., this pressure rises from 60 to 120 kg/sq.m., increasing by 20 kg/sq.m. in the four zones defined as in figure 2a.

Fig. 2a

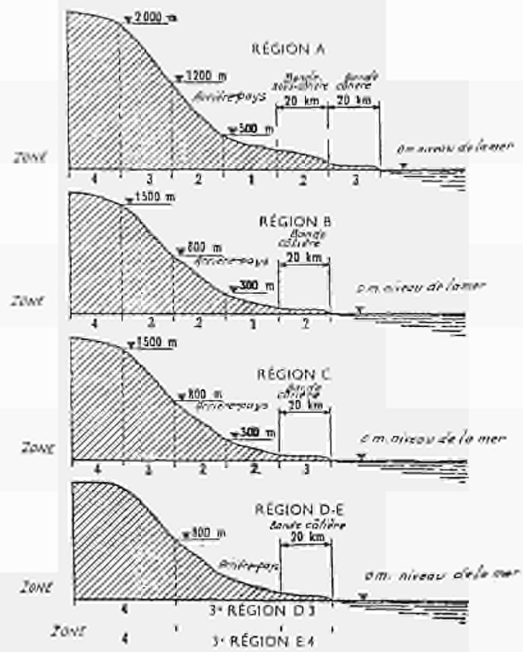

Fig. 2b

— Zone 1 (60 kg/sq.m.) : inland at heights below 500 in Region A and 300 m. in Region B;

— Zone 2 (80 kg/sq.m.) : inland at heights between 500 and 1,200 m. in Region A; between 300 and 800 m. in Region B, below 800 m. in Region C; coastal strip in Region B and subcoastal strip in Region A;

— Zone 3 (100 kg/sq.m.) : inland at heights between 1,200 and 2,000 m. in Region A, between 800 and 1,500 m. in Regions B and C., up to 800 m. in Regions D and E; coastal strips in Regions A, C and D.;

— Zone 4 (120 kg/sq.m.) : inland at heights above 2,000 m. in Region A, above 1,500 m. in Regions B and C, above 800 m. in Regions D and E and coastal strip in Region E.

The coastal and sub-coastal strips and these inlands are defined for the various regions as shown in Fig. 2b. As this illustration shows, the coastal strip includes localities — not screened by a mountain ridge — which are not more than 20 km. away from the sea. The sub-coastal strip includes localities between 20 and 40 km. and limited to Region A. The inland includes localities which are more than 40 km. away in the case of Region A and not more than 20 km. for Regions B, C, D and E.

For the parts of the building exceeding a height of 20 m. above the ground, the kinetic pressure, as determined above, shall be increased by the height H (in metres) as follows (in kg/sq.m.) :

$$\Delta_H = 60\,(H - 20) : 100,$$

but not beyond 48 kg/sq.m. corresponding to a height of 100 m.

As regards temperature variations, barring various exact verifications, a uniform deviation of $\pm$ 30° C. or $\pm$ 20° C. from the local average temperature shall be permissible according to whether the works are more or less directly exposed to weather conditions. Due account shall be taken whenever possible of the difference in temperature between the various components of the same building or between the faces, of the same structure on the assumption that in this case there is a linear variation between one face and the other.

In conclusion, for a quick comparison between the various Regulations mentioned, we can make the following remarks :

(a) The Belgian and French specifications avoid the distinction between main and complementary influences;

(b) The others, including the recent CNR-UNI 10012, maintain this distinction and include among the main influences, those due to dynamics and among the complementary influences, those due to wind, and temperature, also the braking and starting stresses;

(c) The Belgian and French specifications assume three conditions of load. The first condition is practically identical in both specifications. The second includes all the influences, but not allowing for snow in the Belgian specification, but allowing for snow in the French specification. The third, which has an exceptional character, suggests the superimposition of exceptional wind added to the loads of Condition II in the Belgian specification, whilst the French specification, in addition to the permanent load but excluding snow, covers for the worst conditions all extra-loads being increased by 25% and wind being increased by 50 %.

Permissible tensions

NBN 1f 1959 lays down, that for thicknesses of up to 30 mm. and for the loading conditions I, II and III using various grades of rolled steel, the permissible tensions are as in Table I. The data is fixed with reference to the yield point given for each material as shown in the column under Condition III, and as a rule two thirds of its value are deemed to hold good for Condition I and three quaters for Condition II.

Table I — NBN 1 f 1959 = σ_{am} permissible no rmal tension in kg/sq.mm.

Material	Condition		
	I	II	III
A 37 / 37 SC / 37 HS	16	18	24
A 42 / 42 SC / 42 HS	18	20.5	27
A 52 / 52 HS	24	27	36
A 00 Cat. A	14	16	21
A 00 Cat. B	12	13.5	18

These identical ratios are used for thicknesses exceeding 30 mm., for A 52 with $\sigma_S > 36$ kg/sq.mm., but generally meeting the other requirements of the regulations for materials not envisaged in the Regulations, but are considered suitable, when they have a guaranteed average lower yield point or, when the circumstances apply, the limit of 0.2, which is statistically determined, with a typical deviation of results not exceeding 10%.

The safety checks for the various conditions of stress are to be made using the formulas given in Table II. The comparison of the tensions σ_{id}, are written in the usual manner assuming $\sigma_I > \sigma_{II} > \sigma_{III}$, and are derived from the energetic resistance principle of Huber, Hencky, von Mises, which limits the share of afferent specific deformation work to the form variation only. At the triaxial state of traction there is also the limitation $\sigma_I \leqslant 2\sigma_{am}$, as a precaution against brittle fractures. The ideal tension corresponding to triaxial states is also expressed as a function of the plasticity coefficient according to Schnadt, which is dangerously low for a hydrostatic type of condition and equal to one for a monoaxial state. In the flat state of tension expressed by a single tangential parameter, the permissible tangential tension $\tau_{am} = 0.576\ \sigma_{am}$.

Table II — NBN 1 f 1959 — Safety check $\sigma_{id} \leqslant \sigma_{am}$

Stress tension		σ_{id}	
Triaxial tension	principally	$\frac{1}{\sqrt{2}}\sqrt{(\sigma_I - \sigma_{II})^2 + (\sigma_{II} - \sigma_{III})^2 + (\sigma_{III} - \sigma_I)^2} = \sigma_I \pi$	
	generally	$\frac{1}{\sqrt{2}}\sqrt{(\sigma_x - \sigma_y)^2 + (\sigma_y - \sigma_z)^2 + (\sigma_z - \sigma_x)^2 + 6(\tau^2_{xy} + \tau^2_{yz} + \tau^2_{zx})}$	
Biaxial tension	principalally	$\sqrt{\sigma^2_I + \sigma^2_{III} - \sigma_I\sigma_2}$	$(\sigma_{II} = 0)$
	generally	$\sqrt{\sigma^2_x + \sigma^2_y - \sigma_x\sigma_y + 3\,\tau^2_{xy}}$	$(\sigma_z = \tau_{yz} = \tau_{zx} = 0)$
	"	$\sqrt{\sigma^2_x + 3\,\tau^2_{xy}}$	$(\sigma_y = \sigma_z = \tau_{yz} = \tau_{xz} = 0)$
	"	$\tau_{xy}\sqrt{3}$	$(\sigma_x = \sigma_y = \sigma_z = \tau_{yz} = \tau_{xz} = 0)$
Monoaxial tension		σ_I	$(\sigma_{II} = \sigma_{III} = 0)$
Plasticity coefficient : $\pi = \frac{1}{\sqrt{2}}\sqrt{\left(1 - \frac{\sigma_{II}}{\sigma_I}\right)^2 + \left(\frac{\sigma_{II}}{\sigma_I} - \frac{\sigma_{III}}{\sigma_I}\right)^2 + \left(\frac{\sigma_{III}}{\sigma_I} - 1\right)^2}$			

The French Regulations CM 1956 concerning ADx and ADx Charpente give, for the normal tension, the same permissible values — 16, 18 and 24 kg/sq.mm., — which the Belgian Regulations lay down for A 37 in the three similar cases of stress for the two regulations. The complementary safety check, obligatory for a load condition corresponding to the third case, tempers the increases introduced in relation to the figures given in the previous edition of the Regulations. The permissible tangential tension corresponds in all cases to 65% of the corresponding normal tension, as already stated in the CM 46 Regulations.

The safety requirements are satisfied when, in the diagram of the three Mohr circles, representing the state of tension under examination (Fig. 3), points S and S' (at maximum tangential tension or the largest of the three circles) appear not to be outside the shaded zone of the illustration.

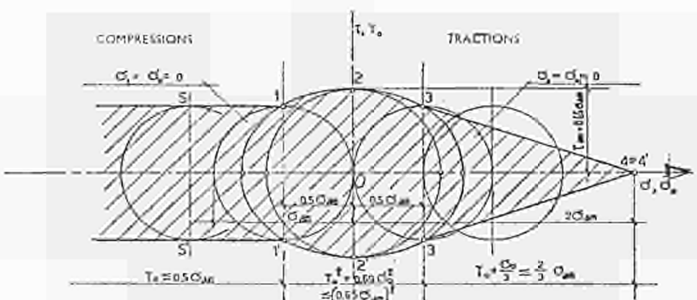

Fig. 3

Assuming the parameters of the larger circle corresponding to a general state of tension :

— the abscissa of the centre : $\sigma_0 = \frac{\sigma_I + \sigma_{III}}{2}$

— and the radius : $\tau_0 = \frac{\sigma_I - \sigma_{III}}{2}$,

The said radius does not cut the axis of the τ through $|\sigma_0| \leqslant \tau_0$, but cuts it through $|\sigma_0| > \tau_0$. At the limit of safety, the compression monoaxial states ($\sigma_I = \sigma_{II} = 0$) and the traction monoaxial states ($\sigma_{II} = \sigma_{III} = 0$) then plot on the contour of the safe zone the pair of points 1, 1' and 3, 3' having ordinate $\tau_0 = 0.5|\sigma_{am}|$; similarly, the two-axial state characterized by a single tangential parameter plots on the axis of the τ the pair of points 2, 2' having ordinate $\tau_{am} = 0.65\ \sigma_{am}$. The limit of safety corresponding to a state of traction of hydrostatic type ($\sigma_I = \sigma_{II} = \sigma_{III}$) obviously expressed by a single point 4 $\equiv$ 4' coresponds to the abscissa, 2 σ_{am}.

This being so, the contour of the safe zone is defined as follows :

— for $$0.5\ \sigma_{am} \leqslant \sigma_o \leqslant 2\ \sigma_{am}$$

by the half straight lines leaving point 4 $\equiv$ 4' and passing through points 3 and 3'; within the said interval the safe zones must consequently satisfy the condition :

$$\tau_o + \frac{\sigma_o}{3} \leqslant \frac{2}{3}\ \sigma_{am}$$

— for $$-\ 0.5\ \sigma_{am} \leqslant \sigma_o \leqslant 0.5\ \sigma_{am}$$

by the ellipse passing through points 1, 2, 3 and symmetrical points 1', 2', 3', to which applies the condition of safety :

$$\tau_o^2 + 0.69\ \sigma_o^2 \leqslant (0.65\ \sigma_{am})^2$$

— for $$\sigma_o \leqslant -\ 0.5\ \sigma_{am}.$$

by the lines passing through points 1 and 1' parallel to the axis of the σ, with the condition :

$$\tau_o \leqslant 0.5\ \sigma_{am}.$$

The application of the method is made easier by a functional scale which, for a given ratio σ_o/σ_{am}, allows to read the ratio τ_o/σ_{am} which must not be exceeded.

For a two-axial state expressed by the parameters σ_x, σ_y and τ_{xy}, we have (Fig. 4) :

$$\sigma_o' = \frac{\sigma_x + \sigma_y}{2}, \qquad \tau_o' = \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right) + \tau_{xy}^2}.$$

If $|\sigma_o'| > \tau_o'$ (σ_x and σ_y of similar sign), the sum $|\sigma_o'| + \tau_o'$ defines the diameter of the larger circle (Figs. 4a and 4c), in which case we must have :

$$|\sigma_o'| + \tau_o' \leqslant \sigma_{am}\ ;$$

if $|\sigma_o'| < \tau_o'$, the larger circle cuts through the axis of the τ, the intermediate principal tension is zero (Fig. 4b) and the condition of safety is expressed by :

$$\tau_o'^2 + 0.69\ \sigma_o'^2 \leqslant (0.65\ \sigma_{am})^2.$$

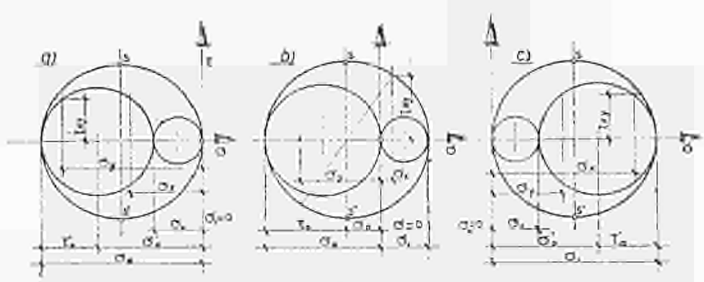

Fig. 4

In particular, for $\sigma_y = 0$, the latter becomes :

$$\sigma_x^2 + 2.36\ \tau_{xy}^2 \leqslant \sigma_{am}.$$

A functional scale gives the pair of values σ_x, τ_{xy} permissible for $\sigma_{am} = 16$ kg/sq.mm. and $\sigma_{am} = 18$kg/sq.mm

For the monoaxial states the specification DIN 1050 lays down different permissible tensions according to whether instability phenomena are to be taken into account or not, quite apart from any special checks which such phenomena might require. The table gives the various values in the two cases and also the values of τ_{am} relating to an isolated tangential stress, for both the condition of load H (main influences) and HZ (main and complementary influences).

The ratio of τ_{am} to the permissible tension for a tensile or compression stress without instability effects is near to the figure of 0.575 to which the Belgian regulations also refer.

Table III — DIN 1050 — Permissible tensions in kg/sq.mm.

Nature of stress	Material			
	St 37		St 52	
	H	Hz	H	Hz
Compression, when the stability check occurs	14	16	21	24
Traction or compression when the safety check occurs (σ_{am})	16	18	24	27
Tangential tension (τ_{am})	9	10.5	13.5	15.5

In the web of compound girders, stresses must comply with the regulation figures contained in the table. Also if the average tangential tension exceeds 0.5 τ_{am}, the ratio of the ideal tension according to Huber, Hencky, von Mises becomes :

$$\sigma_{id} = \sqrt{\sigma_x^2 + \sigma_y^2 - \sigma_x\sigma_y + 3\tau^2}$$

up to the yield point and always be contained within the limits of 3/4 for the condition H and of 4/5 for the condition HZ.

In the case of hyperstatic systems which are designed on the principle of total safety, the ideal local tension cannot exceed the tension σ_{am} of the table.

In sections subjected simultaneously to bending stresses along different axes (M_x and M_y), and possibly in association with a normal force N, the maximum normal tension due to the simultaneous influence of all stresses may be exceeded locally by up to 10% of the regulation permissible value, provided the separate effects of the two bending stresses satisfy the conditions :

$$\max(\sigma_n + \sigma_{mx}) \leqq 0{,}8\,\sigma_{am}, \qquad \max(\sigma_n + \sigma_{my}) \leqslant 0{,}8\,\sigma_{am}.$$

The specification CNR-UNI provides, for monoaxial states and for the initial condition, the permissible tensions of 16 and 24 Kg/sq.mm. for steels of type 1 and type 2, respectively, and tensions equivalent to 1.125 σ_{am} for the second condition. For the multi-axial states, the specification also refers to the principle of Huber, Hencky, von Mises. The safety check is compulsory for both conditions of load.

In conclusion, the Regulations agree in a general way, in respect of materials having comparable mechanical properties, also with the values of the permissible tensions for monoaxial states and the reference to the safety principle given by Huber, Hencky, von Mises for checking the multi-axial states.

There is perfect agreement between the Belgian and Italian Regulations, the latter, however, do not envisage a condition of load corresponding to the third condition of NBN1f. 1959. The provision of DIN 1050, in respect of permissible tensions for compression in given conditions reduced by 10% approximately in relation to those for traction, has no equal in the other Regulations. Specification CM 56, although agreeing on the essential values of common reference, departs considerably from the others owing to the complicated verification method, which, however, meets a more advanced theory and can be easily applied with the help of functional scales given in the Regulations.

The stability of equilibrium

The Regulations concerning the question of stability of equilibrium, and in particular the problem of the bar, either plain or compound, compressed in an axial direction or in a direction parallel to the axis, are the ones which especially differentiate between, and characterize the various Regulations.

Obviously, owing to the shortage of the time available in relation to the vastness and importance of this complex subject, I can only briefly survey the various statements.

NBN 1 f 1959 deals with the combined bending and compression load by reducing it to one of simple compression by means of a suitable reduction of the gross section of the rod. The coefficient of reduction φ, represented by the ratio $\varphi = \sigma_{amp}/\sigma_{am}$ between the permissible tensions for combined bending and compression load and for compression load only, in respect of a given type of steel, is a function of the slenderness of the rod only. The latter is defined as the ratio between the free length, namely the distance between the points of the axis which cannot move transversally, in the bending plane, and the radius of inertia of the perpendicular section in the same plane.

Excluding the combined bending and compression stress for slenderness $\lambda \leqslant 20$, the permissible tension is believed to vary according to the line $\sigma_{amp} = \sigma_c/\nu_c = a - b\lambda$ a plastic field ($20 < \lambda \leqslant \lambda_P$), according to Euler's hyperbola $\sigma_{amp} = \pi^2 E/\nu_{CE}\lambda^2$ in the elastic field ($\lambda \geqslant \lambda_P$). The degree of safety ν_c is the same of simple compression, whilst the ν_{CE} is constant and equal to 2.7 as suggested by Prof. Massonnet. The constants a and b, are different for the various materials, and are such that $\sigma_{amp} = \sigma_{am}$ for $\lambda = 20$ and σ_{amp} equal to the Eulerian pressure divided by the degree of safety, to the limit of proportionality. The latter has been taken to be equal to 80% of the yield point, which for $\sigma_s = 24$. 27 and 36 kg/mm for A 37 (or A 00), A 42 and A 52 respectively, given in the same order $\lambda_P = 105$. 98 and 85.

Enclosure III of the Regulations contains the values of the coefficient, which are different for the three grades of steel. The values adopted for other kinds of materials must be justified theoretically and confirmed experimentally. The use of slenderness $\lambda > 175$ is left to the responsibility of the designer, but in general is not recommended. The coefficients φ of the table hold good for all three types of Regulation loads, it being assumed that the σ_{amp} in the case of types 2 and 3 can be increased in the same ratio as for simple compression.

If the compression stress does not act along the axis, or generally when in association with it there is also a bending stress, the verification for maximum tensions corresponding to the superimposition of both stresses is prescribed. The compression stress is evaluated as in the case of the axial load. The bending stress must be increased by the ratio $P_E/(P_E - 2.7\ P)$, with P_E, being the Eulerian load for head-on yielding in the corresponding plane, in order to take into account the increase in bending moments due to deflection. Also a further increase, in the ratio 1 : K, taking into account the risk of twist, is to be added to the bending stress in the plane of maximum inertia, K being the coefficient laid down for that purpose in Arts. 426 and 427 of the Regulations. Consequently, in the more general cases, the condition to be satistied corresponds to the formula :

$$\sigma = \frac{P}{\varphi A} + \frac{M_{xeq}}{KW_x\left(1 - 2.7\frac{P}{P_E}\right)} + \frac{M_{yeq}}{W_y\left(1 - 2.7\frac{P}{P_E}\right)} \leqslant \sigma_{am}.$$

If the bending moment is highest at one end of the rod, as when the compression load acts on the axis, the calculation moment in each main plane is to be found by the formula :

$$M_{eq} = \sqrt{0.3\ (M_1^2 + M_2^2) + 0.4\ M_1M_2}$$

in which the moments $M_1 = Pe_1$ and $M_2 = Pe_2$ at the ends of the rod play their part in value and sign. If M_{eq} is lower than max M, the check for the compound bending may be more important and consequently determinant :

$$\sigma = \frac{P}{A} + \frac{M_x max}{W_x} + \frac{M_y max}{W_y} \leqslant \sigma_{am}$$

The Règles CM 1956 deal with the problems of stability by systematically reducing them to a compound bending stress; it is known that the method takes into account, not only the geometrical imperfections, but also the faults in the material.

The check condition of the simple compressed rod follows the usual formula $\sigma k_1 \leqslant \sigma_{am}$, in which σ is the average tension, σ_{am} is the permissible tension for a simple compression and k_1 is the increase coefficient which takes into account both the theoretical and practical aspects of the combined bending and compression stress and is given in Table IV of enclosure 7.1 of the Regulations, in function of the ratio $\mu = \sigma_{CR}/\nu\sigma$ of the Eulerian pressure and the average pressure multiplied by the degree of safety.

The average pressure can also be related to the permissible tension σ_{amp} which takes into account the combined bending and compression stress by simply making $\sigma \leqslant \sigma_{amp}$. The comparable tension, obtained by dividing the tension at the yield point by the degree of safety, is given in Table III of the above-mentioned Enclosure, for slendernesses of 20 to 300 and for values of $\sigma_{am} = 16$ and 18 kg/sq.mm. corresponding to to the yield point $\sigma_s = 24$ kg/sq.mm.

For the rod loaded under axial compression and subjected to a bending stress in the axial compression plane, the following two cases are important :

(1) Symmetric or asymmetric section, with a bending stress σ_f due to compression in the maximum stressed fibre of ordinate $v \geqslant v'$ (Figs. 5a, b);

(2) Asymmetric section with a bending stress σ_f due to compression in the maximum stressed fibre, of ordinate $v' \leqslant v$ (Fig. 5c)'

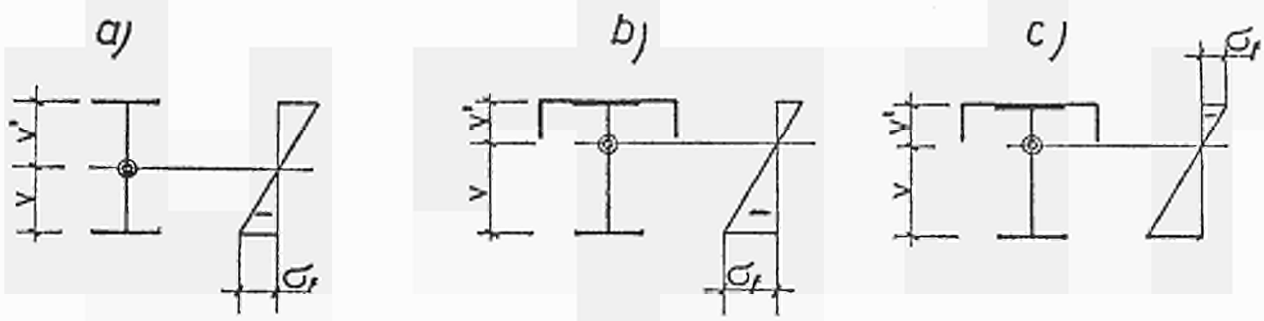

Fig. 5

In the first case, the safety condition is a single one :

$$\sigma k + \sigma_f k_f \leqslant \sigma_{am} (1 - 0.2\ k_o)\ ;$$

in the second case, the two following conditions must be satisfied :

$$\sigma + \left[\sigma (k - 1) + \sigma_f k_f\right] \frac{v'}{v} \leqslant \sigma_{am} \left(1 - 0.2\ k_o \frac{v'}{v}\right),$$

$$\sigma (k - 2) + \sigma_f k_f \leqslant \sigma_{am} (1 - 0.2\ k_o).$$

The coefficients k, k_f and k_o are found, as a function of the ratio μ, in the same Table IV of the Regulations, mentioned in connection with the coefficient k, which occurs in the first method for checking the rod simply compressed.

DIN 4114 lays down, for the plain rod axially compressed, the familiar check based on the so-called Omega method :

$$P\omega/A \leqslant \sigma_{am}\ ;$$

in evaluating P, account must be taken of possible dynamic and suitability factors and not of the fatigue effect.

The coefficients ω, formally similar to the coefficients k_1 of CM 1956, have here the expression $\omega = \sigma_{am}/\sigma_{amp}$ in which σ_{amp} is equal to the smaller of the following ratios :

— σ_{CE}/ν_{CE} : Eulerian pressure $\sigma_{CE} = \pi^2E/\lambda^2$ / degree of safety $\nu_{CE} = 2.5$.

— σ_{CR}/ν_{CR} : effective critical pressure / degree of safety $\nu_{CR} = 1.5$.

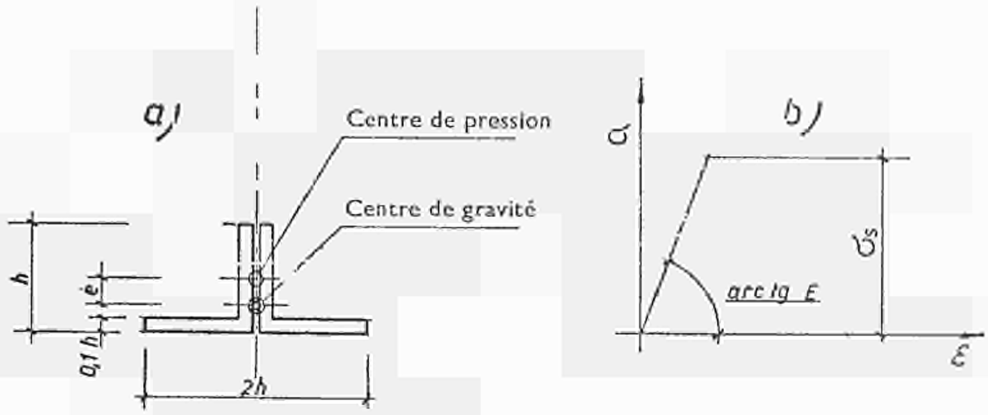

Fig. 6

As regards the evaluation of σ_{CR}, it is assumed that the rod is articulated at the ends, and (a severe assumption) that it has a section as shown in Fig. 6a with the compression load, (behaving in a straight line which does not vary with deformation,) and acts according to the eccentricity :

$$e = (0.05 + \lambda/500)i,$$

as shown in the illustration. So that in relation to the radius of bending inertia *i*, the eccentricity consists of a constant amount and also of an amount proportional to the slenderness λ. The classical theories of deflection are deemed to be satisfied even beyond the limit of proportionality, the deformed section is reduced to a sinusoidal section and the assumption is made that : E = 21,000 kg/sq.mm., $\sigma_S = 23$ kg/sq.mm. for St 00 and St 37 and $\sigma_S = 34$ kg/sq.mm. for St 52. Under these very rigid assumptions, if we have :

$$m = 2.317\ (0.05 + \lambda/500),\ \rho = m\sigma_{CR}/(\sigma_s - \sigma_{CR}),$$

the effective critical tension σ_{CR} is identified by the equation :

$$\sigma_{CE}\left(1 + \frac{m}{\rho}\right)(1 - \rho + 0.25\,\rho^2 + 0.005\,\rho^3) = \sigma_s = \text{const.}$$

To obtain the σ_{amp} relating to the second type of load, it is evident that we must widen those of the first type in the ratio $\sigma_{am2}/\sigma_{am1}$.

For slendernesses increasing from $\lambda = 20$ to $\lambda = 250$, Table I of DIN gives the coefficients ω for St 00.12 and St 37.12, and Table II those relating to St 52.

For the rod loaded under axial compression and bent in one of the principal planes of inertia, using an approximate method, two cases are important in these Regulations according to whether the ratio of the ordinate v of the compressed edge to v' of the drawn edge due to the bending effet v/v' is greater than, equal to or smaller than :

(1) for $v/v' \geqq 1$ (Figs. 7a, b), is sufficient the check :

$$\frac{P\omega}{A} + 0.9\,\frac{M}{W} \leqq \sigma_{am}\ ;$$

(2) for v/v' $<$ 1 (Fig. 7c), both the following conditions must be verified :

$$\frac{P\omega}{A} + 0.9\,\frac{M}{W'} \leqslant \sigma_{am},$$

$$\frac{P\omega}{A} + \frac{300 + 2\lambda}{1000}\,\frac{M}{W} \leqslant \sigma_{am}.$$

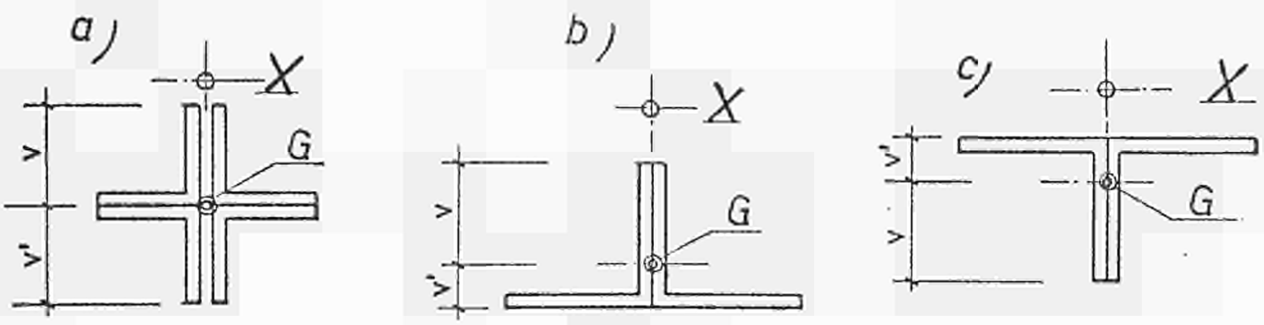

Fig. 7

When the bending moment varies along the axis, whilst the stressing plane remains the same, under the conditions mentioned above we must put in max M in place of M; however, if the maximum coincides with one of the moments M_1 and M_2 acting at the ends and in correspondence with them the deformed section has ordinate zero, we can take an average value $M = 0.5\,(M_1 + M_2)$, but not less than 0.5 max M.

The specification CNR-UNI 10011 deals with the axially compressed load of the plain rod following in general lines specification DIN 4114, adopting also, for the type of load 1 and for both grades of steel envisaged, a constant degree of safety $\nu = 2.50$ for $\lambda \geqslant \lambda_p$, namely for slendernesses in an elastic field, and decreasing in a plastic field, up to a value of $\nu = 1.50$ corresponding to $\lambda = 0$. Owing to the various basic permissible tensions for the two types of steel, namely 14 and 21 kg/sq.mm. in DIN and 16 and 24 kg/sq.mm. in CNR-UNI, the latter gives coefficients ω higher than 14% approx., but equally onerous.

For the rod subjected to a combined pressure and tension stress, if an easier check is not required, the following condition is made to apply :

$$\frac{P\omega}{A} + \frac{M\omega_1}{\left(1 - \frac{P}{P_E}\right) W} \leqslant \sigma_{am},$$

in which P_E is the Eulerian load, assumed to be valid also in the plastic field, and ω_1, is a coefficient which allows for the risk of twist — similar to the coefficient K of the corresponding formula in the Belgian Regulations — and is given by special tables, as a function of a non-dimensional ratio depending on the geometric characteristics of the rod and separately for the two grades of steel. For determining M in the case of a variable bending moment, the similar principles of the German Regulations apply also.

For a summarized comparison of the various Specifications, strictly confined to the subject considered, we can point out that for the classical experimental behaviour suggested by Tetmajer in the plastic field all regulations (excepting the Belgian ones) have substituted more complex critical pressure/slenderness laws, whilst at the same time facilitating their application through the use of suitable tables.

We cannot say that it is always appropriate to vary the degree of safety in the useful field of slenderness, a problem which seems to have been rationally solved in CM 1956.

As regards to the economy in the design of compressed rods, which considerably influences the general economics of structural steel constructions, the French Regulations seem the more advantageous whilst the Belgian Regulations, (in which the maximum degree of slenderness currently utilized is very restricted,) the most reliable, and the German and Italian Regulations are in the intermediate position.

Conclusion

It is very difficult to draw conclusions from the overall review that has been made and from the few remarks which I have made to follow each subject. As I said in the introduction, my exposition had to leave out entirely, some essential aspects, both static and technological, and I should have been delighted to have dwelt on some of them which were particularly studied whilst elaborating on the Italian Regulations, one of which for example, was welding connections. Moreover, as regards the subjects I have dealt with, I have had to confine myself to putting them forward, whilst leaving out details which were far from being unimportant, in order to give a well founded opinion.

The conclusion, therefore, can only be given in a general manner.

As regards the definition of constructional steels, the discussion is still open.

The principle of limiting the materials to only two classes sufficiently qualified, namely one standard and one high-strength, seems now to be general practice. In this selection, special steels do not come in, for example, the American type T 1, so far not taken into account by any of the European Regulations.

As regards the stress influences, there is justification for some of them, due to climatic or traditional causes, for the different approaches from one Regulation to another, but it is not always easy to find a reasonable explanation for the very marked differences which are found in the case of others. It is indisputable that it should be most appropriate to try to reach a better understanding on the corresponding evaluations.

On the other hand, satisfactory agreement seems to exist on the two types of stress; there is hope for proof in relation to a third type — exceptional — as laid down in the Belgian and French Regulations.

There are no notable differences as regards the level of the basic permissible tension, or the degrees of safety. In this connection, it must still be decided whether, taking into account the range of the yield point with the variation in thickness, it would be advantageous, for a given material, to adopt a single permissible tension, thus accepting the corresponding ranges of the degree of safety or whether it would be logically preferable to tolerate the complication of a more moderate permissible tension for thicknesses beyond a certain limit.

The principle of Huber, Henckey, von Mises for checking the safety of multi-axial stress states is generally accepted. The criticism against it is not without foundation, nevertheless as the principle of the traditional check of safety has been accepted, the former principle has proved satisfactory, being confirmed by experience and represents the best that at present can be done.

The stability of equilibrium is acquiring ever increasing importance whilst the properties of the material improve and constructional forms become more slender. It sets complex problems which often involve difficult research methods, but of which the solution is not always in agreement with the experimental results. In any case it does not lend itself well to being expressed in a single and quick manner as appropriate to Regulations. In this connection, it seems to us that the French Regulations are not only the most progressive, but they also have given a rational complete and final solution to these complex problems.

The various types of connections, *i.e.* rivets, standard and high-strength bolts, welds — have been left out of my exposition. However, I should like to make a very brief reference to them in order to underline the growing importance, for many applications, of the behaviour of fatigue in joints. The specification CNR-UNI 10011 has given adequate space to this subject, especially in connection with welded joints. A large programme of research on joint fatigue for tubular steel work for various types of joints, especially using friction bolts, has been put under way on behalf of the International Committee for the Development of Tubular Structural Steelwork (C.I.D.E.C.T.), by the Structural Steel Construction Centre of Pisa University. It is expected that a start will be made as soon as the preliminary static tests, now running, have been completed and that the work will be finished next year.

The Structural Steelwork Construction Industry has derived already enormous advantages from the European Coal and Steel Community. Interest in it, which was formerly limited to a small circle of specialists, has considerably increased and is quickly increasing further. The same interest is noted in its applications, ever so frequent and important, often in large works and also in fields traditionally used to constructional types in other materials.

These results are already encouraging, but we can be certain that we shall see more promising developments if the efforts aimed at reaching a European Specification are to be crowned with success.

Problems and Trends in Steel Construction from the American Angle

(Original text: English)

George E. DANFORTH

In the Fall of 1960 I had the great pleasure to present a series of lectures on the Continent and in the United Kingdom on recent developments in the architectural uses of stainless steel in the United States and Canada. Within the following two years I gave a similar series to architects in Japan, Australia and India. As I consider such occasions significant forums for the exchange of information and as opportunity for presenting ideas and concepts which hopefully may be conducive to provocative thinking and further constructive development, I was particularly honored by the High Authority of the European Coal and Steel Community in being invited to participate in this, the first, international Steel Utilization Congress.

Of course being a student of Mies van der Rohe I am more than a little interested in the problems of structures as a factor of architectural expression — notably steel structures.

The program formulated for this Congress impresses me as being a sound beginning to what I hope will be a continuing contribution to an international understanding of the problems of architecture, building and construction. Being an architect I will of course attempt to touch upon some of the problems and trends of the use of steel in America with more or less architectural examples.

But what are some of these problems and in what ways do they present themselves? In my talks with architects, fabricators and builders in many parts of the world I have been impressed by the generally similar nature of the problems with which they are concerned. But this is not surprising, for slowly, within the past century, architecture has come to accept modern industry and technology as the source of materials and methods of building. This impact of the scientific and technological age in which we live has resulted in a uniformity of architectural expression never before realized on so universal a scale; but the understanding and acceptance of its relationship to architecture was one of the first and most important problems which the architects and builders of America had to face — in short — one of a philosophy of building, not rooted in historical revivalism, but one leading to a new scale of values in which industry and industrial products would be the sources out of which a new and inspiring art would flourish and grow.

The seeds for this new creative architecture resulting from the new technology of the Industrial Revolution found root in the work of William Le Baron Jenny when he invented the steel skeleton in the Home Insurance Building in Chicago in 1883 (1). The outward expression of that skeleton was unclear and lost its great force in uncertainty, however, its significance was in its implication of a new constructive order, as indiginous to the time as was the rib vaulting of the gothic cathedral.

Jenny's invention of the steel skeleton eliminated the necessity of bearing wall construction and made possible the development of the tall building. The impracticability of the bearing wall for high buildings was made clear in the Monadnock Building of 1889 in Chicago by John Wellborn Root where the exterior masonry walls were almost seven feet in thickness at thgraound floor. Nevertheless the Monadnock Building and the Reliance Building done in 1894 by Daniel H. Burnham are prime examples of one of the truths implicit in the work of the Chicago School Architects of the late 19th century — that truth being that a building itself, contains *within* itself the creative quality (2 and 3).

But this simple fact was probably the most difficult for American architects to understand. The new concept of the steel skeleton was an embarrassment to them, and they could use it only as a prop for applied historical styles; merchandising the buildings in a packaging which conformed to the current fashion. In America, a new country, and, therefore, a country largely without any restraining tradition, the architectural results were shocking. The effect of this short-sightedness lasted until the late 1930's when we find in the work of Mies van der Rohe an architecture projecting the ideas implied in the work of the early pioneers such as Sullivan, Jenny, Burnham and Root (4).

Looking back to Mies van der Rohe's early projects of the 1920's, which along with those of Le Corbusier, and Gropius, to mention a few, proclaimed a new vision, one is impressed today by a characteristic common to all of them, that is, a strongly objective spirit. The new movement, of which Mies was a leader, spread across all national boundaries and swept away the excess of personal expression and subjective fantasy which had preceded it (5).

Architecture was not to be, in short, concerned with personal expression as such, but the problem of architecture itself was to be stated simply by examining, in a new light, a very old question — what is Architecture ?

As one of the great pioneers of the new architecture none more clearly revealed the influence of an objective spirit than Mies van der Rohe. This spirit was intensified and developed until the recent work of his mature years is strongly marked by a clear and objective discipline. In fact it becomes increasingly clear that his architecture is nothing less than the development of an idea. It is Mies' unique contribution to have seen that behind form lies structure as a basis of form, and this insight has led him to the idea of a structural architecture. It is also one of his characteristic contributions to be able to see and express the severe, precise, and linear character of steel, and to make of it an architecture, at one and the same time, of power and refinement.

In Mies' first building in America, the Metals and Minerals Research Building (1941) at the Illinois Institute of Technology, he deliberately selected the elements of building from the already existing products of an advanced technology, and used them with care and consideration in developing a structural type (6). Actually the structural type itself is not in reality an invention, but rather the refinement and clarification of recognizable structural types, such as the skeleton in steel as so clearly expressed in his Chicago apartment buildings done in the early 1950's. Exhausted concepts of form originating in traditional building materials and methods are no longer in conflict with new structural concepts. Utmost resolution of elementals has been achieved in these buildings and at the same time fullness of sensuous harmony. They draw upon their own inexhaustible resources, extending anew our aesthetic sensibilities (7 and 8).

Probably one of the most significant buildings in the development of steel construction in America is Crown Hall, done by Mr. van der Rohe on the campus of the Illinois Institute of Technology in Chicago in 1956. This great building houses the School of Architecture, Planning and Design and is the architectural emphasis of the entire campus design. Its unique structural concept is that of an all-steel and glass structure permitting the main floor to be entirely free of columns (9).

At the dedication of Crown Hall, the late Eero Saarinen said that... "it is the best expression of a whole culture that is growing out of a technical civilization—", and continued by saying that Mies' buildings are built on one great principle — that architecture should be a part of our time and that the technology of our time should be expressed in every building from the over-all concept to the smallest detail (10 and 11).

It is precisely because of his concern with fundamentals of building that the recent works of Mies van der Rohe have such great meaning for our time.

Two basic structural concepts have occupied his attention for some time past : the skyscraper or multi-storied building, such as in the Chicago apartment buildings just seen or the Federal Building Complex now in construction in Chicago, and the hall of free span and of great dimensions. This latter type found its most forceful expression in his project for a convention hall in Chicago (1954). Over 700 feet square the great two-way truss is supported only on its periphery at 100 foot intervals permitting a completely free area within for the multipurpose functions such a hall must accomodate (12, 13 and 14).

Whether we examine the perfection of detail, or survey the simplicity of overall form, we are cognizant of a far-sighted intelligence at work, which has accepted as its goal, no less than the task of formulating for our time, the „arch-types" of a new architecture in steel.

Mention of Mies van der Rohe's work is prerequisite to any discussion ot the use and development of steel in buildings in America because of the pioneering he has done in bringing this material out into the open-as it were. Remarkable too was this early American work, not only because it suggested and clarified an entirely new philosophical approach to the ae thetics of structure, but that so much of it could be accomplished in the face of inadequate and often antiquated building codes, limited knowledge of appropriate fabricating and erection techniques, and excessive engineering design standards, to mention a few.

This is not to infer that in some of these areas we have progressed too far from the situation as it was in the late 1940's. To the contrary.

One instance is in the field of architectural research. In spite of the good intentions of many of our manufacturers, architectural research is, I believe, too frequently confused with the development of a manufacturer's product; too often applied to the service of special interests. I can think of no serious study having been made of contemporary building problems which involved the task of bringing about a harmonious relation between applied science and technology, and our social needs.

With a material such as steel, which lends itself so well to sub-assembly, we still do not include in our quasi-prefabricated structural systems those mechanical and electrical components which today comprise up to 30 or 40 percent ot the total cost of a building.

Why hasn't the building industry solved the problem of its electrical systems as has been done in the electronic industry or the airplane industry? Until a wall, floor or roof building component can serve not only as a structural and space enclosing element but also as a source of heat and light we are working with outmoded concepts in relation to the real potential of our time.

The ultimate use of steel in building construction in the United States has been further limited by the slow development of protective coatings against fire. No one, to my knowledge, has seriously questioned the code restriction of a four hour fire rating (why four hours?) nor questioned the logic behind the restriction which permits an architect to enclose a structure with glass (which has little or no fire rating) but imposes a fire rating on noncombustible metal panels if used as a curtain wall. For a country known for its uniform standards we, in fact, have no real common specification or performance standard. Again, research might well be directed towards a more critical examination of the very principles underlying our national building restrictions.

If the universality of manufacturing, fabrication, and construction processes — which are implicit in our industrial and technological age — can be achieved, such groups as The High Authority of the European Coal and Steel Community can be an instrumental force in bringing about more international cooperation in defining and establishing building and construction standards.

It would be fruitful, for example, to devote time to an objective study of the fundamental differences between the engineering design philosophies of Europe and America. Open competition in design in Europe as well as European construction codes seem to encourage greater innovation than in America where we demand such elaborate — and to some minds — excessive safeguards to protect the design. Has this been a reason why the knowledge and particularly the acceptance of orthotropic design is so far behind in the United States?

Economic factors of course strongly enter into the problem what with the greater ratio in Europe between the cost of the material and the wage scale. But of course this too is changing : the ratio between the two factors is getting smaller in Europe, and in the United States the economics of steel construction has radically changed for the better within the last two years. This latter is due, primarily, to the numerous advances in the properties of tectonic steels and also new techniques for their application.

Dr. John B. Scalzi, structural engineer with the United States Steel Corporation optimistically states in an article "Spectrum of Steels" published in the September 1961 issue of *Progressive Architecture* that, "Never before in the history of the steel industry have there been so many new steels, steel products, diversified fabricating techniques, and steel design concepts at the call of the architectural profession." Elsewhere in his article Dr. Scalzi notes that construction steels now range in yield point from 33,000 psi to a yield strength of 100,000 psi. For the engineer this makes possible a much more efficient design through savings in the weight of the material (thus less cost), simplified fabrication, reduced maintenance, lower shipping costs, easier handling and erection and less fireproofing because the individual members are smaller.

One of these new high strength, low-alloy steels has an added characteristic which enhances its use where exposed to atmospheric conditions. One of this particular type of steel, USS Cor Ten, is four to six times better than carbon steel in its resistance to corrosion, because its surface oxidizes thus forming a tightly adherent film over the material which in turn prevents further oxidation. Besides its obvious applications to industrial use where high strength, minimal maintenance is desirable it has recently been used in some major architectural projects, two of which I would like to show you.

The first full architectural use of Cor Ten, is the new administration building for the John Deere Company, Manufacturers of farm and road building machinery. Built near Moline, Illinois it was one of the last buildings done by Mr. Eero Saarinen, before he died (15).

The building situated in the rolling countryside near the Mississippi River, is strong in its structural expression with the intention, according to the architect, of symbolizing the heavy, sturdy character of the machinery for which the John Deere Company is so well known (16).

After having been exposed to the weather for approximately two years the patina is approaching that rich, deep brown which will characterize its final color (17).

The present complex consists of the major eight story administration building and a smaller structure, housing exhibitions and meeting halls, which building is connected to the office unit by a Vierendeel Truss bridge (18 and 19).

The tracery formed of the steel sun screens and the strong structural elements of the building is, in certain light, sharply accented by the laminated, heat resistant reflective glass used throughout the project (20).

The second building is a major structure in more than one sense. It is the new Civic Center Courthouse now under construction in Chicago, Illinois. When completed it will be the tallest building in the city, rising over six hundred and sixty feet (31 stories) and its 48 by 87 feet bays are the largest structural bays in any building yet designed for actual construction. These large spans and the floor to floor heights of 18 feet provide space necessary for what will eventually be 162 courtrooms and hearing rooms (21).

C.F. Murphy Associates are the supervising architects, with Skidmore, Owings & Merrill and Loebl, Schlossman & Bennett as associated architects. Mr. Jacques Brownson, of the C.F. Murphy office, and a former student of Mies van der Rohe is the designer. It is therefore no accident that the character of the Civic Center building has the strong structural quality so evident in Mies' work.

The building is monumental in scale, but not overwhelmingly so. It is a proud building in the tradition of the Chicago School of Architecture, in which work, decades before, it has its roots.

The Civic Center building with its attendant plaza to the south occupies an entire city block in the heart of Chicago's business district. It is a building employing a large percentage of high strength steels with Cor Ten being used exclusively on the exterior (22).

The columns are stepped and cruciform in shape, arc welded and made of ASTM A-441 type steel (Fig. 1 and 2). At the lower floors they are five feet across, stepping down to about two feet at the upper stories.

What with the fireproofing and the exterior skin of Cor Ten the columns at the ground level measure approximately 6 feet 4 inches (23).

The interior framing members are open stringers with sprayed on light weight fireproofing, and the exterior framing is a composite of stringers and girders — six feet in depth, and of 3/8 inch thick Cor Ten. On the back of each girder steel lugs serve to bond it to the concrete fireproofing and thence to the open web stringer.

The oxidation process of Cor Ten can be seen in photographs 24 and 25, with the lighter almost gunmetal color being on the most recently placed plate girders.

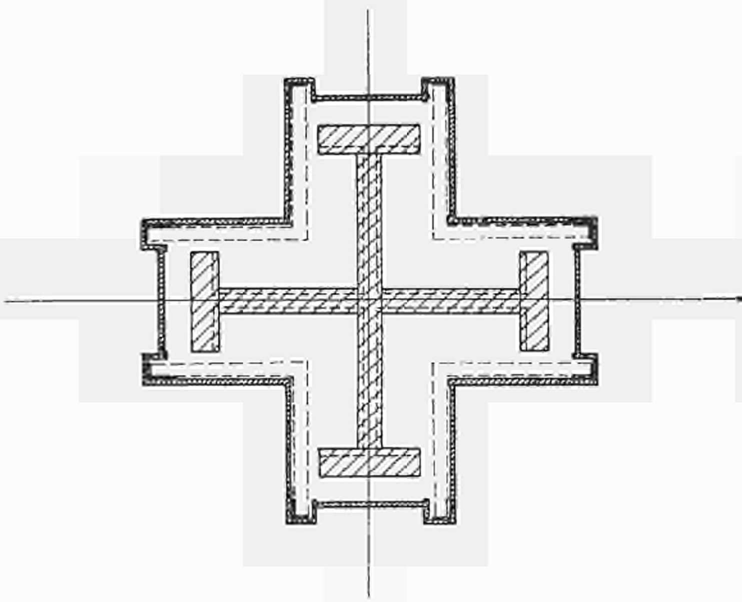

Fig. 1

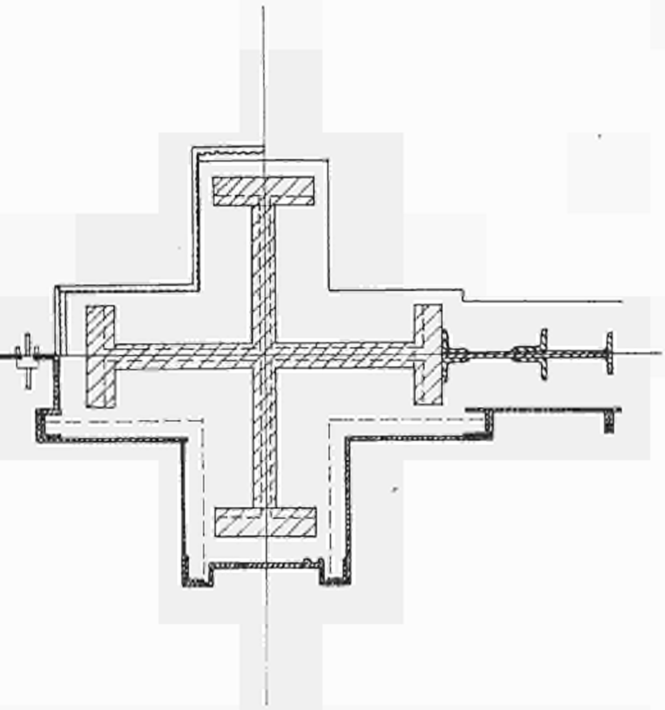

Fig. 2

Incidental to the architectural qualities of the high strength steels and to the use of Cor Ten and other like oxide coated materials which will undoubtedly have a much wider use in the future is the 27,000 tons of framing steel and 4,500 tons of exterior architectural steel (totalling 31,500 tons) being used in the Civic Center building.

The structural concept of the International Business Machine (IBM) Building recently completed in Pittsburgh, Pennsylvania is that each of the four enclosing wall planes is a diagonal grid truss, sheated in type 302 rigidized stainless steel with the floor beams spanning 54 feet from the exterior wall truss to a central service core. The interior spaces are thus completely free of columns. Each wall truss is supported at the base by only two trussed columns. There are no supplemental framing members in the walls other than the basic diagonal and horizontal members of the truss (26, 27 and 28).

The engineers fully employed the high strength steels where stresses made them applicable and during the construction, before the stainless steel sheathing was applied, the individual truss members were painted in color symbols indicating the three basic structural steels used : the A 36 being an improved carbon steel with a minimum yield point of 36,000 psi, the A 441 with a yield point near 50,000 psi and the Heat Treated Constructional Alloy steel having a minimum yield strength of 100,000 psi. Although this was primarily an advertising device it was none the less interesting to observe the distribution of the primary and secondary stresses in each wall truss (Fig. 3).

Such a structural concept, not withstanding its debatable architecture merit — interior as well as exterior — seems to portend more widespread use in view of the fact that the structural engineers preliminary designs indicated the selected diagonal grid system used as a load bearing wall would require 250 tons less steel than conventional post and beam construction, primarily because the steel diagonals work only in tension and compression, instead of in bending.

Using the outer wall as a truss of rectangular elements, Minoru Yamasaki & Associates and Emery Roth & Sons have adapted this aforementioned structural concept to the proposed two 110 storey towers of the World Trade Center in New York (29). These towers each 1350 feet high and 209 feet square will, as in the IBM building, have each interior floor space free of columns by virtue of using 60 feet by 13 feet prefabricated floor sections. These span the 60 feet between the stainless steel clad exterior rectangular bearing wall grid and the interior service core (Fig. 4).

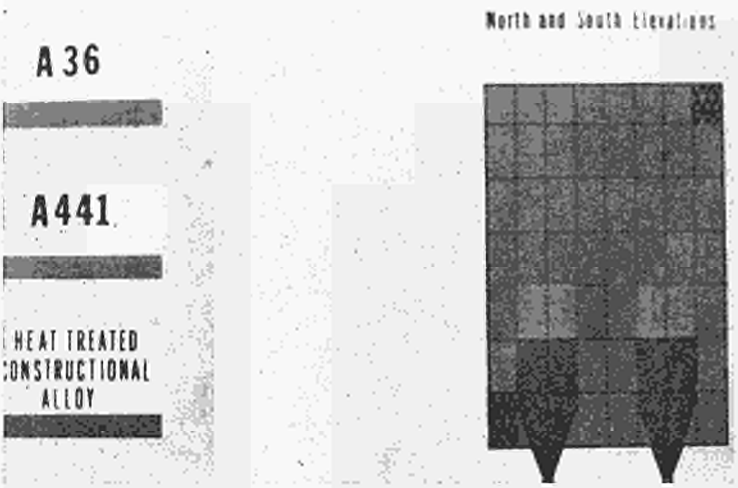

Fig. 3

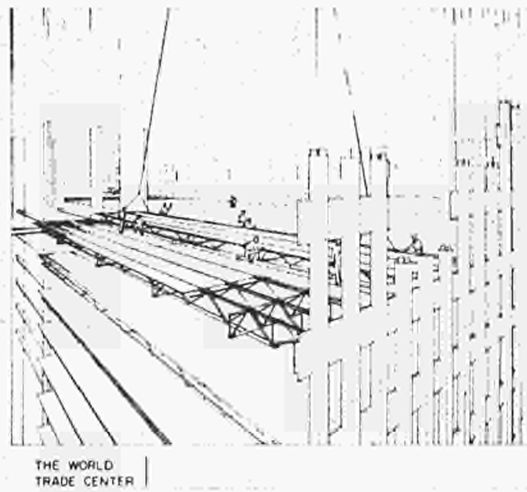

Fig. 4

But not all of the new buildings in America are so vertical in their dimensions. There is the need to provide large, column free spaces for such activities as sports arenas, meeting halls, industrial complexes, etc. For this, entirely different structural systems are being used, providing enclosures of a magnitude unfeasible before the Industrial Revolution and the advent of steel.

Pittsburgh, Pennsylvania's Public Auditorium (30) is 415 feet in diameter, 136 feet high and its enclosing steel and stainless steel clad dome was designed with moveable sections permitting the auditorium to be about 50% open when desirable. The moveable wedge-shaped sections pivot at the apex of the dome and ride on a track system at the base.

In Houston, Texas, there is being completed the Harris County Stadium. This Lamella dome is 641 feet and 8 inches in diameter and is covered with a new lucite material developed by Dupont which allows the passage of ultra violet rays and still serves as a heat retardant (31).

Still another system is the Geodesic dome for the Union Tank Car Company in Baton Rouge, Louisiana. Here the steel Geodesic dome is structurally integral with the carbon steel roofing. Its diameter is 384 feet (32 and 33).

Not so architectural in the sense of the previous examples but of great interest to engineers as well as architects is the construction of the Gateway Arch in St. Louis by the late Eero Saarinen. Mr. Saarinen conceived the structure as an inverted catenary curve, the strongest configuration for an arch, with all the thrust passing down through the legs into the foundation. It is a stressed skin structure in which prefabricated triangular sections are placed one on the other, welding the inner and outer walls together and filling the "sandwich" with concrete. More specifically the inner steel walls are of 3/8 inch carbon steel and the outer walls of 1/4 inch stainless steel. The concrete within the two walls is post tensioned with high strength alloy steel bars. The crest of the arch is 630 feet above the ground and the spread at the base approximately 315 feet (34 and 35).

Other than in the great bridges of the world little use has been made of the cable suspension system to enclose space architecturally (36). Examples such as the United States Pavilion at the Brussels World Fair or in the restaurant at O'Hare International Airport in Chicago far from realize the great expressive power inherent in the suspension principle.

Its great force is indicated in this project done by a member of the faculty and students in the Department of Architecture at Illinois Institute of Technology in Chicago. The span is 2,000 feet and is achieved by combination of the steel cable suspension system and the two way roof truss. The limits of the project were to explore the architectural qualities inherent in a system of structure appropriate to the enclosure or covering of a vast area. Still to be resolved are the problems of wall enclosure, space divisions and mechanical and electrical considerations (37 and 38).

Such a project does however bring to focus in all its aspects the very problems that face us today. How can we build with the intent of using our technology in its fullest sense, and using our judgment in the application of that technology with the result that what is done is expressive of our day and only of our day.

In closing it seems appropriate to this thesis, to quote from Mies van der Rohe :

> "I believe that architecture has little or nothing to do with the invention of interesting forms, or with personal whims. I believe architecture belongs to the epoch, not to the individual. That at its best, it touches and expresses the very innermost structure of the civilization from which it springs. Greek temples, Roman basilicas, and medieval cathedrals are significant to us as creations of a whole epoch rather than as works of individual architects. It must be understood that all architecture is bound up with its own time; that it can only be manifested in living tasks and in the medium of its epoch. In no age has it been otherwise."

Description of photographs

1 — Home Insurance Building, Chicago.

2 — Monadnock Building, Chicago.

3 — Reliance Building, Chicago.

4 — Building in New York.

5 — Glass building by Mies van der Rohe.

6 — Metals and Minerals Research Building, Chicago.

7 - 8 — Chicago apartment buildings: under construction, and viewed from the lake.

9 - 11 — Crown Hall Building: façade, interior view and projecting steel column.

12 — Model of Federal Building Complex, Chicago.

13 - 14 — Convention Hall: steel skeleton and exterior view.

15 - 20 — John Deere Company Administration Building, Moline, Illinois.

21 - 25 — Civil Center Courthouse, Chicago: model, plaza' column in profile, plate girder, general view.

26 - 28 — International Business Machines Building (IBM): under construction, columns and erection view.

29 — Model of World Trade Center, New York.

30 — Pennsylvania Public Auditorium, Pittsburgh.

31 — Harris County Stadium, Houston, Texas.

32 - 33 — Union Tank Car Building, Baton-Rouge, Louisiana: exterior and interior views.

34 - 35 — Gateway Arch, St. Louis: model and construction view.

36 — Mackinao Bridge.

37 - 38 — Project by the Illinois Institute of Technology of a suspension system.

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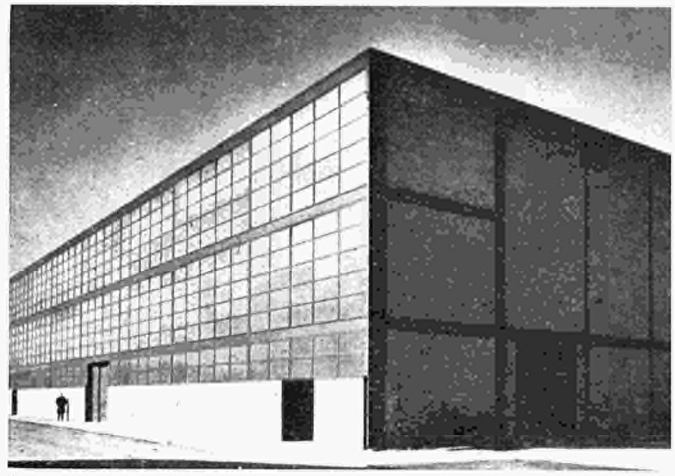
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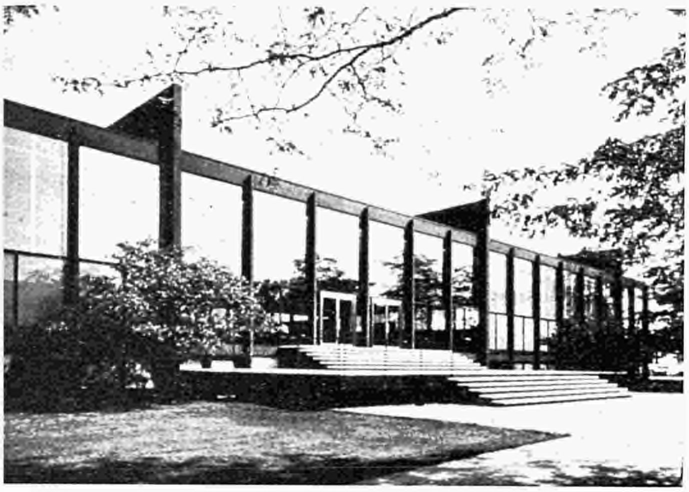

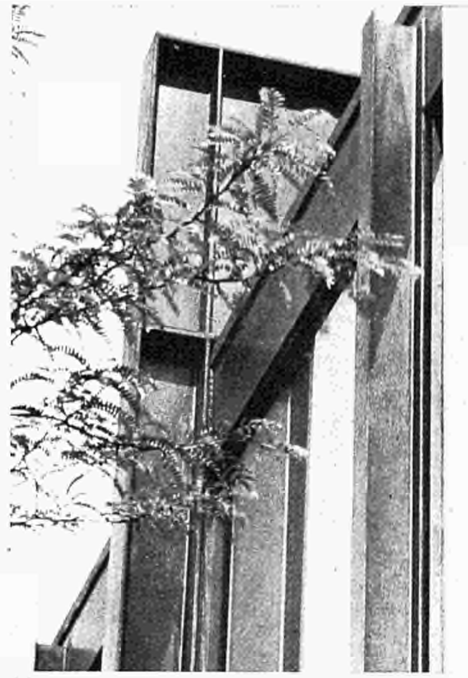

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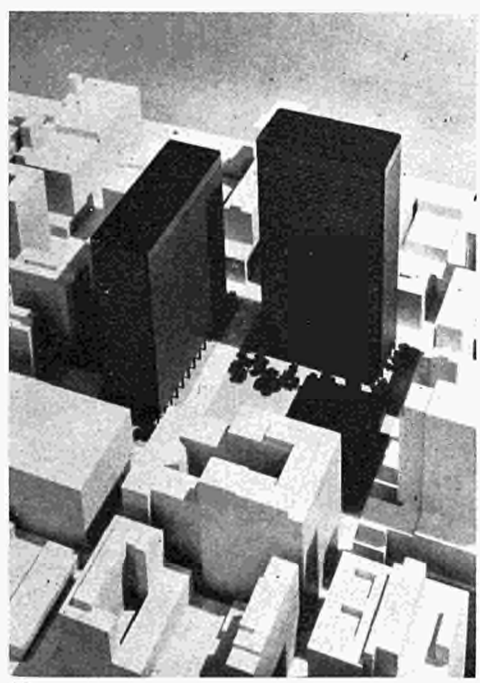

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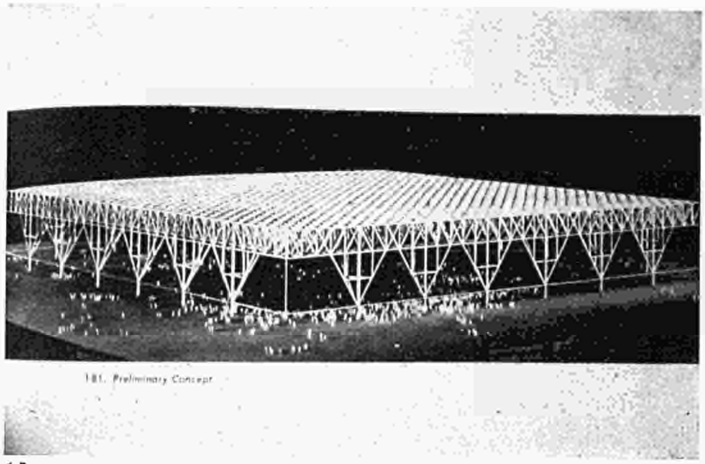

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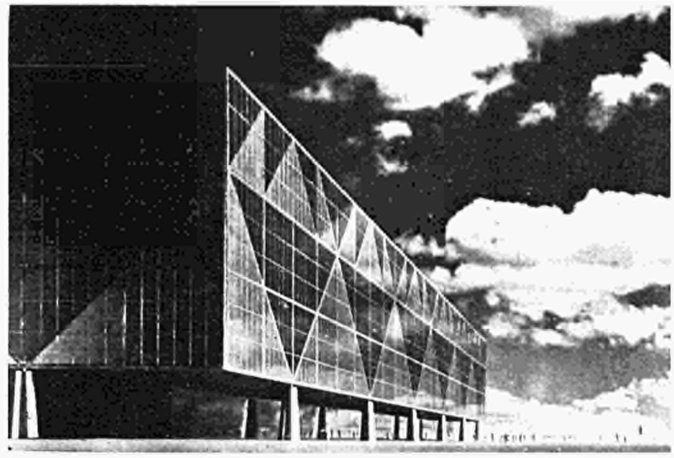

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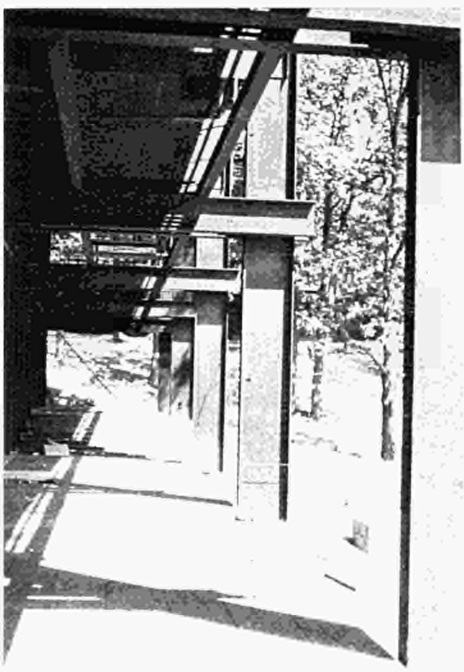
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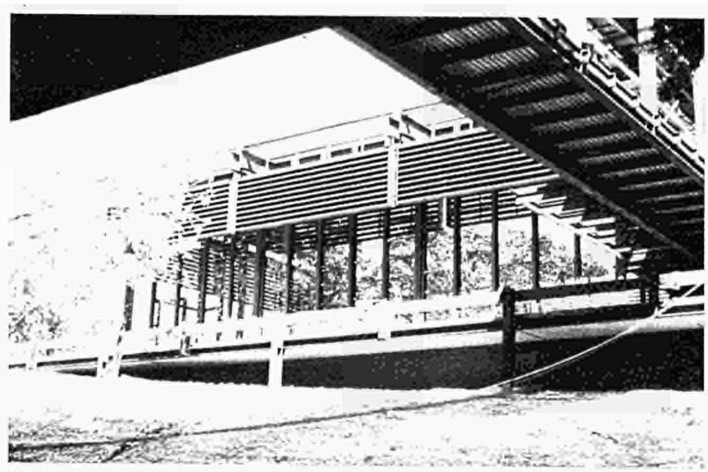
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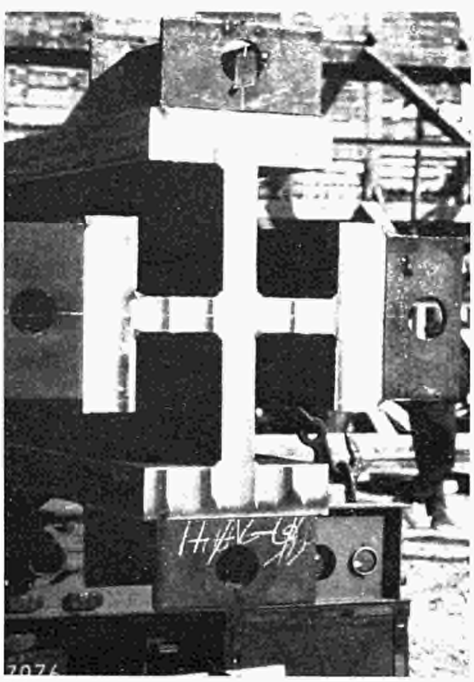
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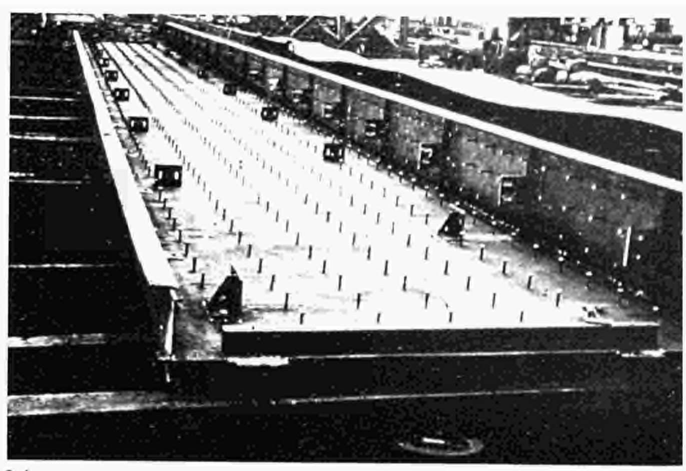
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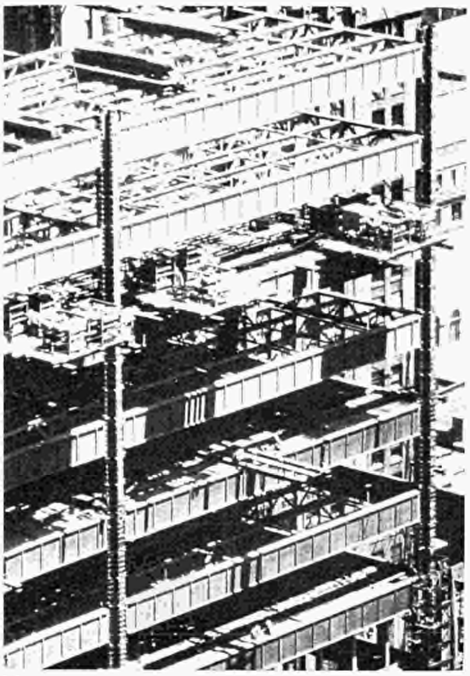
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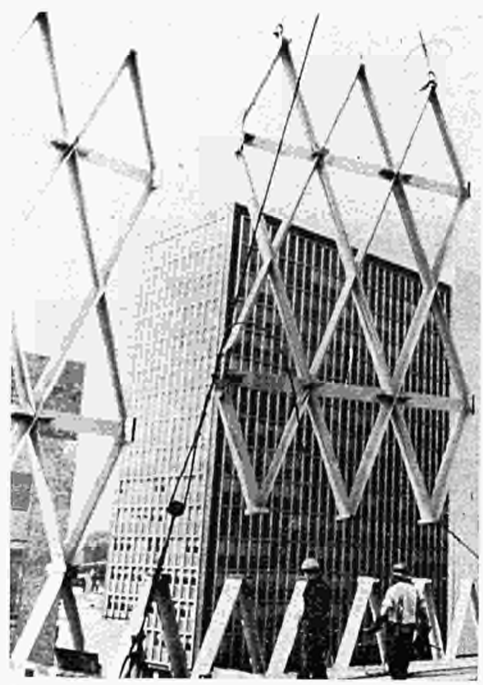
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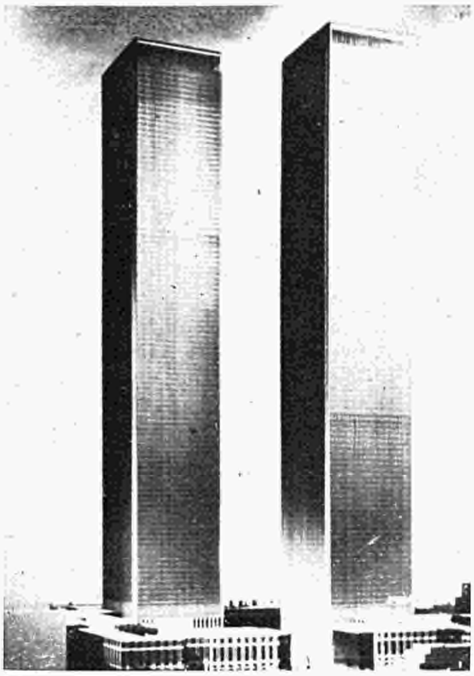
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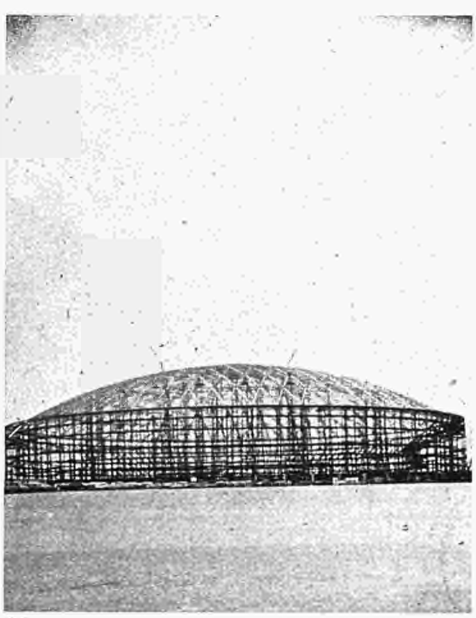
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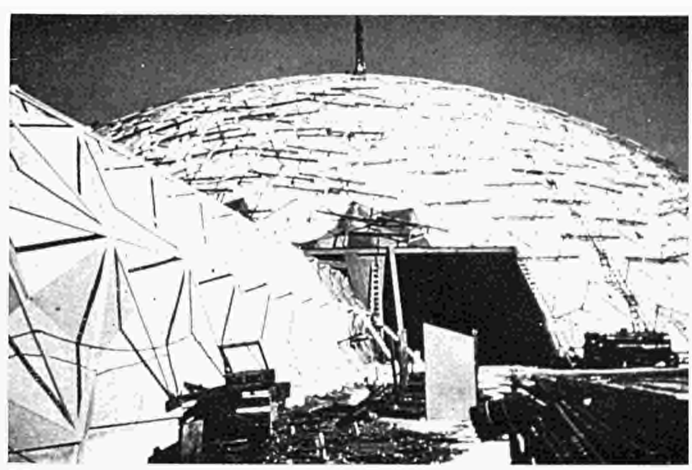
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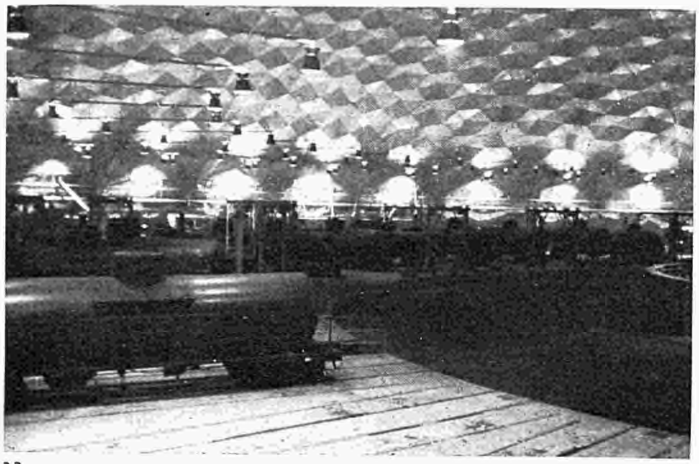

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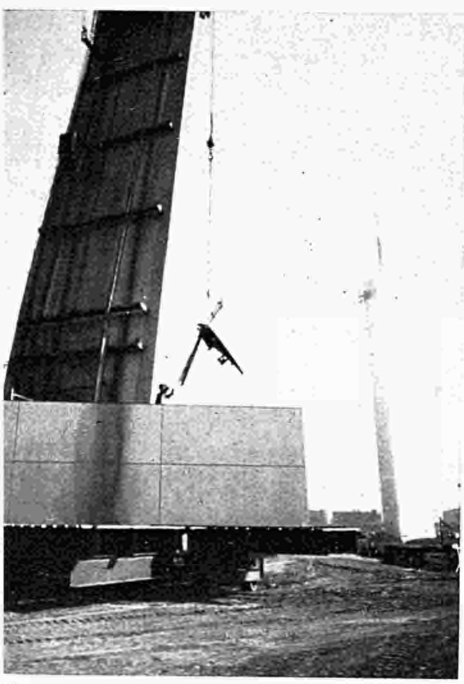

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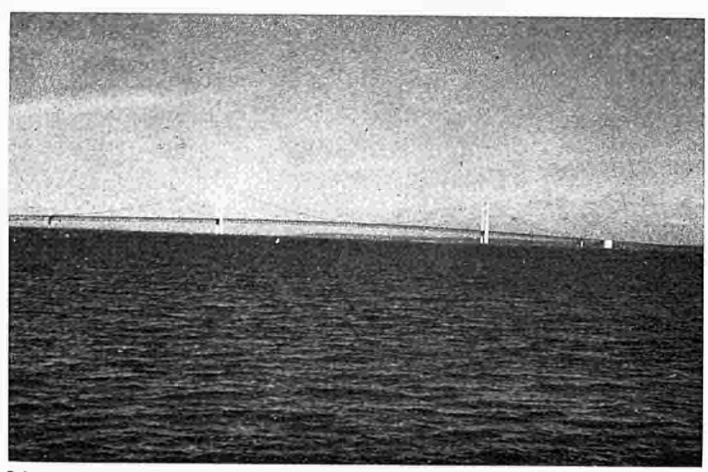

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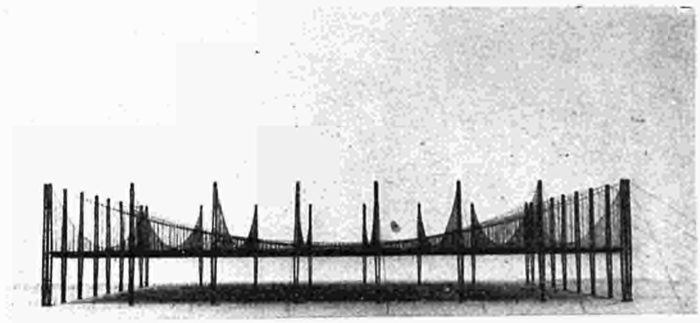

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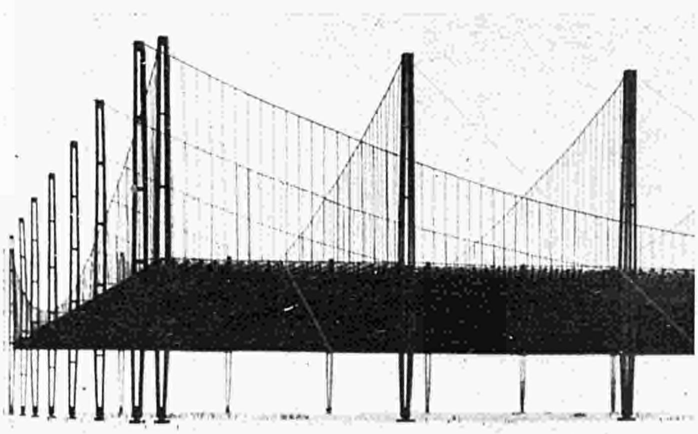

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3 *Proceedings of the Working Parties*

CONTENTS

WORKING PARTY I :

Brigdes, Elevated Roads and Flyovers

Chairman:

Dr-Ing Giorgi BARONI

Rapporteurs:

Prof. Hiroshi KIHARA

H. SHIRLEY SMITH

The Working Party dealt with the subject of bridges, elevated roads and flyovers.

Many aspects were touched on by the rapporteurs and speakers in the discussions. The rapporteurs' papers described a number of interesting designs and projects relating to the construction of steel and long-span bridges, particularly in Japan.

Several speakers drew attention to the need for fuller study on

(a) prefabricated components and sections and their assembly connections;

(b) weldability of high-yield-point steels;

(c) rolled products and production runs thereof.

Points particularly brought out included the study of the forms most suited to steel, as calculated to make the most of its special qualities, and the recent development of a girder in welded stainless sheet which has lately aroused much interest in technical circles in the United States.

The Working Party's proceedings confirmed the suitability of steel for use in bridges, elevated roads and flyovers.

Hiroshi KIHARA

Toshie OKUMURA

Present Status of Steel Bridges in Japan

(Original language: English)

Introduction

It may be said that, evaluating the bridges built in Japan, an important rôle played by the welding technique should not be overlooked. In 1932, the first application of welding to the reinforcement of the „Hiyamagawa Bridge" (deck type plate girder) had been conducted and followed by construction of nearly 10 welded bridges, for example, the "Tabata Bridge" (span 40.5 + 53.0 + 40.5 m.) in 1935.

After World War II, the all welded Gerber Bridge, „Egawa Bridge" [1] was erected in the Hiroshima Prefecture in 1949 and after the completion, loading- and vibration tests were carried out to ascertain the performance of this all welded bridge. [2] In 1951, the real all welded bridge „Honkyu Bridge" [3] in the Hyogo Prefecture was constructed. In this construction, flanges and webs were first separately buttwelded into one complete piece and then tack-welded in a specially designed assembly frame to secure an exact "I" shape. Finally, welding had been done conforming to an adequate welding sequence in a rotatory welding frame.

This assembly method was successfully adopted to fabricate welded plate girders as well as truss-members in comparatively less man-hours. The abovementioned welding method together with the performance and safety of welded bridges, — ascertained by loading tests and vibration tests —, was taking the place of the rivet joint system. All joining work in shops has been done by welding since 1955. Thus, in order to meet the increasing demand of the construction of steel bridges, new types of steel girders such as composite girders, box girders, orthotropic steel plating, Lohse girders and curved girders, have successively been made by welding.

Present status of steel bridge fabrication

Figure 1 shows the increasing annual expenditures for steel and concrete bridges erected in Japan by the governmental expenditures from the budget. The numerals of expenditures — including not only those of superstructures, but also those of substructures and of the others, give the qualitative image on the tendency of bridge production. The remarkable increasing tendency of steel bridges depends on the facts that adoption of welded steel bridges makes construction time shorter and economical efficiency higher.

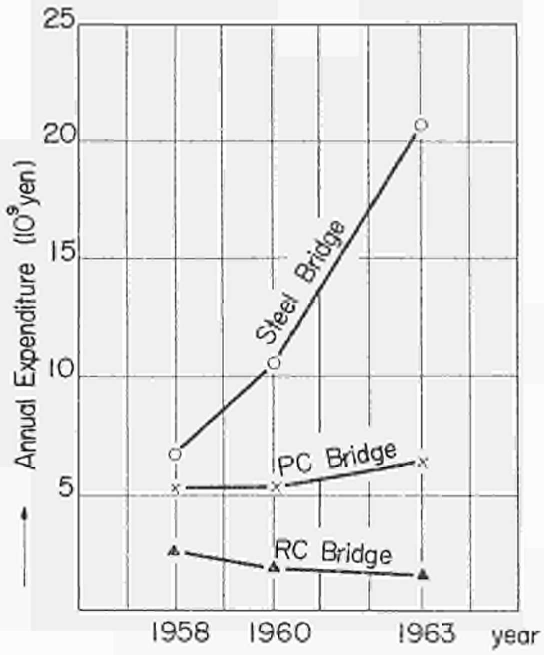


Fig. 1

Recently it is commonly recognized that a bridge forms a link in the chain of traffic roads. A bridge was in the customary way first designed and then the roads are planned to suit the bridge. Nowadays, road planning between two places is firstly drawn and then a bridge is designed so as to meet the alignment of the road. Consequently the economical efficiency of the bridge is more or less neglected in order to keep the high traffic efficiency as a link in the chain of traffic road. This trend has been accelerated by the construction of expressway road systems.

Moreover, the following facts make a contribution to increase the share of steel bridges in road planning, i.e. the establishment of theoretical calculation methods for curved bridges (as will be described later), progress in welding techniques, weight saving and economy due to adoption of high yield strength steels, and shorter fabrication time. It should be noted that approximately 40% of the whole length of the expressway in Tokyo built by the Tokyo Expressway Public Corporation is equipped with steel bridges, although steel bridges are apt to be used for the construction of expressways in large modern cities such as Tokyo and Osaka.

The demand for R.C. (Reinforced Concrete) bridges is now decreasing because this type of bridge is usable only for short spans, and therefore limited to local road bridges and because it needs rather long construction time and higher girder-height compared with those of other types of bridges.

With regard to P.C. (Prestressed Concrete) bridges, the demand has not grown as expected. This is due to the fact that the production capacity of P.C. Bridge fabricators is almost completely filled by orders. Considering, however, that the P.C. method has progressed technically and various new methods for P.C. have been developed, it is expected that the need for P.C. bridges will increase further in the future, in spite of the many existing difficulties such as supervision in the field due to the lack of capable inspectors.

Fig. 2 shows the volume of the production of steel frames (building) and steel bridges during the last 10 years. (1954 to 1963). This figure clearly proves that steel frame work seems to reach its saturation point, but, on the contrary, steel bridges still go up rapidly. In 1963, the production tonnage of steel bridges increased 293·1%, compared to that in 1960, and in the same period only 116·3 % in steel frame works.

The fabrication cost of steel frame works is about yen 83,000 (Bfr 11,600) per ton in 1963, against yen 155,400 (Bfr 21,750) per ton for steel bridges; which is nearly twice that of steel frame works and three times that of the steel price per ton.

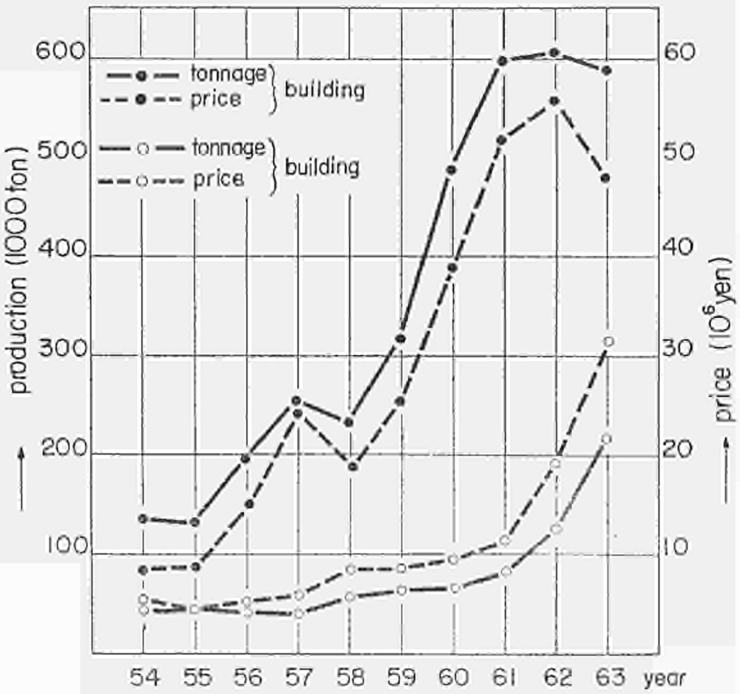


Fig. 2

According to the survey of the Steel Rib and Bridge Association, the shop fabrication-cost per ton of various types of steel bridges, such as welded plate girder, composite girder, truss, box girder and arch bridge, are shown in Table I. The standard man-day/ton of these types of girders made of S.S.41 (tensile strength $\sigma_B \geqq 41$ kg/mm², yield strength $\sigma_Y \geqq 23$ kg/mm²) and S.M. 50 ($\sigma_B \geqq 50$ kg/mm², $\sigma_Y \geqq 32$ kg/mm²) are shown in Table II. As can be seen in Table II, welding takes about 25-30 % per man-day, while the assembly stage requires a rather high percentage of man-days which could be much reduced by improving assembly processes.

Table I

Prime Cost of Steel Bridge *

	The use rate of SM 50 (%) (SS 41 for the rest)	Steel (yen) **	Fabrication (yen) **	Total cost (yen) **
Plate Girder	0 14 63	61,859 58,504 68,861	61,735 62,214 64,606	123,594 120,718 133,467
Composite Girder	0 25 71	61,859 61,397 71,612	69,513 70,948 73,819	131,372 132,345 145,931
Truss	0 29 55	61,582 62,087 71,079	75,490 77,883 79,318	137,072 139,970 150,397
Box Girder	0 35 70	61,820 60,625 69,879	77,883 79,318 81,711	139,703 139,943 151,590
Arch	0 29 42	59,249 65,726 64,660	87,574 89,967 91,402	146,823 155,693 156,062

* Costs for packing, transportation, erection, management and profits are excluded, so business price would be higher than these costs by 10-20%.

** Yen 100 = \$ 0.27

Table II — Standard man-day for Welded Highway Bridge (man-day/ton)

	Case of SS 41						Case of SM 50					
	Machining	Assembly	Welding	Temporary Erection	The others *	Total	Machining	Assembly	Welding	Temporary Erection	The others *	Total
Plate Girder	1.5	2.8	3.3	0.3	3.0	10.9	1.6	2.8	3.6	0.3	3.1	11.4
Composite Girder	1.5	2.0	3.9	0.6	4.5	12.5	1.6	2.1	4.2	0.6	4.6	13.1
Truss	2.1	2.5	3.8	1.1	4.6	14.1	2.3	2.7	4.0	1.1	4.6	14.7
Box Girder (Orthotropic Steel Plating Excluded)	2.2	2.5	4.0	1.2	4.4	14.3	2.5	2.5	4.2	1.2	4.6	15.0
Arch Girder	1.0	2.3	4.1	2.5	6.5	16.4	1.4	2.5	4.3	2.5	6.6	17.3

* Full-scale Marking, Rolling, Drilling, Gas cutting, Forging, Riveting etc. are included.

High yield strength steels in Japan

The greater part of steels used for bridges in Japan consists of S.S.41 ($\sigma_S \geq$ 41 kg/mm², $\sigma_Y \geq$ 23 kg/mm²) and S.M.50 ($\sigma_S \geq$ 50 kg/mm², $\sigma_Y \geq$ 32 kg/mm²). These two steels are specified in J.I.S. (Japan Industrial Standard) which is a Japanese National Standard. S.S.41 is an ordinary mild steel and S.M.50 is a weldable high tensile steel corresponding to German S.T.52.

Various kinds of high yield strength steels have been widely adopted in many fields of Japanese industries, while few high yield strength steels are specified in J.I.S. The Japan Welding Engineering Society established W.E.S. (Welding Engineering Standard) which includes a high yield strength steel specification and this specification has been widely accepted in Japan. This specification[1] (W.E.S.) classifies high yield strength steels into 9 classes as shown in Table III, where two Arabic numerals behind H.W. mean proof yield strength (kg/mm²) of the steels. So that, T-I steel (U.S. Steel) and H.Y.-80 (U.S. Navy) correspond to H.W. 70 ($\sigma_Y \geq$ 70 kg/mm²) and H.W.56 ($\sigma_Y \geq$ 56 kg/mm²) respectively.

Table III — Specification for High Yield Strength Steel (WES)

Grade	Yield Point σ_Y (kg/mm²)	Tensile Strength σ_S (kg/mm²)	Standard V — Charpy Test: Plate Thickness t (mm)	Testing Temperature T (°C)	Mean Energy of 3 Specimens E (kg—m)	Maximum Equivalent Carbon Ceq (%)	IIW Maximum Hardness Test H max. (VHN)
HW 36	≥ 36	53 ~ 65	13 ≤ t < 21	10	≥ 4.8	0.48	380
			t ≥ 21	0			
HW 40	≥ 40	57 ~ 70	13 ≤ t < 21	5	≥ 4.8	0.49	390
			t ≥ 21	— 5			
HW 45	≥ 45	60 ~ 72	13 ≤ t < 21	0	≥ 4.8	0.50	400
			t ≥ 21	— 10			
HW 50	≥ 50	62 ~ 75	13 ≤ t < 21	0	≥ 4.8	0.54	415
			t ≥ 21	— 10			
HW 56	≥ 56	68 ~ 82	13 ≤ t < 21	0	≥ 4.8	0.58	430
			t ≥ 21	— 10			
HW 63	≥ 63	74 ~ 85	13 ≤ t < 21	— 5	≥ 4.0	0.60	440
			t ≥ 21	— 15			
HW 70	≥ 70	80 ~ 95	13 ≤ t < 21	— 10	≥ 3.6	0.62	450
			t ≥ 21	— 20			
HW 80	≥ 80	88 ~ 105	13 ≤ t < 21	— 10	≥ 2.8	0.74	470
			t ≥ 21	— 20			
HW 90	≥ 90	97 ~ 115	13 ≤ t < 21	— 15	≥ 2.8	0.80	490
			t ≥ 21	— 25			

* $Ceq = C + \frac{1}{6}Mn + \frac{1}{24}Si + \frac{1}{40}Ni + \frac{1}{5}Cr + \frac{1}{4}Mo + \frac{1}{14}V$

Usually, steels having a yield strength of 45 kg/mm² or less, are "As Rolled" or "Normalized" and steels having a yield strength of 50 kg/mm² or more, are heat treated or "Quenched and Tempered". In order to increase yield strength and to improve notch toughness of steels at low temperature, special heat treatment (so-called IN treatment) which gives rise to proper precipitation of aluminum nitride is sometimes conducted. In the case of very high yield strength steel such as H.W.70 or more, there were several difficulties to be solved, for example, problems on brittleness of welded joints at bond; at heat affected zones; and on sulphur and hydrogen cracking due to H_2S. These difficulties have steadily been solved by many active distinguished researchers.

The above mentioned 9 classes of high yield strength steels have widely been accepted and put to use for the right high yield strength steel in the right places. In the field of bridge costruction, H.W.50 ($\sigma_Y \geqq 50$ kg/mm²) as well as S.M.50 ($\sigma_S \geqq 50$ kg/mm²) are wiedely used, and this will be described later. Now the authors would like to introduce a brief note on the application of high yield strength steels in other fields of industry in Japan.

In the shipbuilding industry S.M.50 and H.W.50 are commonly used for ship's decks and bottom plates, sheer strakes, angled, and round gunwales, deck and bottom girders, deck and bottom longitudinals, hatch side girders, bulkhead top plates etc., with the result that 2000-4000 tons of high yield strength steels are used for a ship and 400-800 tons of hull-weight saving is achieved.

With regard to pressure vessels: the increasing demand for L.P. Gas reaches up to 2,000,000 ton/year in Japan and a lot of storage tanks for L.P.G. have been constructed. During the last 5 years, more than 150 storage tanks, capacities of which are about 300 m³- 3,000 m³, have been built of high yield strength steels, mainly of H.W.50. H.W.56 and H.W.70. H.W.63 ($\sigma_Y \geqq 63$ kg/mm²) has been used for penstock of electric power stations and H.W.90 for pressure vessels in the chemical industry. In this way, the various fields of industry have a tendency to use more and more higher yield strength steels.

The Japan Welding Engineering Society gives a steel manufacturer of a new high yield strength steel only then an approval when all the test-results of the steel satisfy the W.E.S. specification of high yield strength steels and additional requirements. Therefore, it is no exaggeration to say that only high yield strength steels approved by the Society are sold to steel fabricators and users in Japan. Some of these high yield strength steels, having an excellent notch toughness at low temperatures, may also be classified by the W.E.S. specification of steels for low temperature use[5], as such.

In this case, the Japan Welding Engineering Society may give to the steel another approval as a steel for low temperature use, which specifies the lowest working temperature of the steel. In other words, certain types of steel may have two approvals, one for high yield strength steel and another for low temperature use.

Use of high yield strength steels in bridges

Since 1954, when S.M.50 was used for the first time for the "Sagami Bridge", the amount of S.M.50 used for bridges shows a rapid increase and it is found that the share in weight of S.M.50 is almost half of the total weight of steels used for 130 newly constructed bridges. The ratio of S.M.50 to S.S.41 used for each bridge, of cource, depends on circumstances and is determined theoretically from the economical point of view.

For instance, because of easy erection, good appearance, clearance and the location of the approach, it is often necessary to make girder height equal all over the span of a bridge, neglecting more or less reduction of rigidity of the bridge. In these cases, plate thickness should vary so as to follow the bending moment distribution. Consequently when heavy plate is required, high yield strength steel plays an important rôle to overcome the difficulties resulting from secondary stresses and notch toughness of heavy plate. In these bridges, generally S.S.41 is 60-40*W* and S.M.50 is 40-60*W* in weight. These percentages, of course, depend on the span length and steels used.

Recently, H.W.50 (W.E.S. $\sigma_Y \geqq 50$ kg/mm²) is widely adopted in long span plate girders with constant girder height. In this connection Table IV and Fig. 3 illustrate the case of the recently completed Meishin. Expressway (from Nagoya to Kobe). In the Expressway, continuous composite girders were adopted. The ratio of S.S.41 to S.M.50 used in the Expressway is 6:4 where the span is 30m., while 5:5 where the span is 40m. and H.W.50 is used. When the span is wider than 40m., the most economical ratio becomes S.M.50 = 60%, H.W.50 and SS41 = 20% each. It may be said that the features of plate girders mentioned above together with the tendency of wider span, increase the demand of high yield strength steels and require higher yield strength steels. In the case where the span is rather long and a variable girder height is allowed by the situation of the site, continuous variable section girders, curved Warren truss bridges and arch bridges are adopted in this order, according to the length of the span. The rigidity of these bridges is usually very high, compared to that of a bridge with equal girder heigth.

The use of high yield strength steels for the above mentioned bridges has two main reasons; the first is simply to reduce the dead weight of the bridge and in this case bridges, entirely built of high yield strength steel are doubtlessly the most suitable, and the second is to use high yield strength steel, only for the parts of stress-concentration, since ordinary mild steel is good enough to bear the stress produced by the dead weight. So, in the former case the greatest part of the steel weight of the bridge consists of high yield strength steel and in the latter case of mild steel. It is, therefore, natural that with regard to the curved Warren truss bridge and the recently constructed arch bridge, the ratio of weight of high yield strength steel to the total weight is about 70% (the former case) or 20% (the latter case).

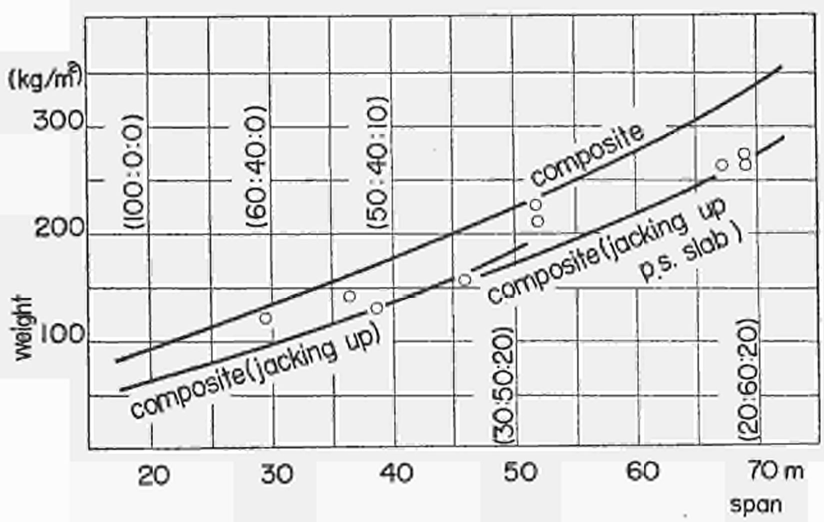
As regards the use of H.W.50 in bridges, as stated above, rationalization of structural design by using H.W.50 had been done in case of the Meishin Expressway (see Fig. 3 and Table IV). Figure 4 shows the increasing amount of H.W.50 used by the Tokyo Expressway Public Corporation. Especially to overcome the difficulties in design of the elevated road system in Tokyo, caused by the narrow and limited space in the city and to solve the high stress concentration factor at the corner connections of rahmen structures, high yield strength steels are quite effective.

Table IV — Bridges erected in Meishin Expressway.

Name of Bridge	Span * (m)	Composite and Pre-stress	Girders × height (m)	Weight, (the use ratio) ** (kg/m) %	Remarks
Enmyoji	3 c 21.8	none	4 × 1.44	121 (100 : 0 : 0)	Curved Bridge
Taga over rail road	18.35 + 22.5 + 18.35	»	5 × 1.00	119 (37 : 63 : 0)	Limitation to Girder height
Yasuigawa	24.0 + 28.0 + 24.0	»	4 × 1.50	139 (100 : 0 : 0)	
Obatagawa	2 c 29.6	jacking up at support	4 × 1.20	125 (63 : 37 : 0)	Limitation to Girder height Skew Bridge
Inukamigawa	31.0 + 2 c 42.5 + 31.0	»	3 × 2.00	140 (34 : 66 : 0)	
Serigawa	35.0 + 46.5 + 35.0	»	3 × span 1.60 support 2.80	132 (44 : 43 : 13)	Emphasis on Appearance
Hiogawa	40.0 + 48.0 + 40.0	none	4 × 2.20	200 (83 : 17 : 0)	
Mukogawa	3 c 46.6	jacking up at support	3 × 2.00	155 (34 : 45 : 21)	
Yasugawa	3 c 52.2	jacking up to support post tensioned concrete slab	4 × 2.10	210 (23 : 64 : 13)	Curved Bridge
Yasugawa	2 c 52.2	»	4 × 2.10	225 (23 : 64 : 13)	Curved Bridge
Kisogawa	3 c 67.3	»	4 × 2.70	264 (18 : 62 : 20)	
Ibigawa	5 c 69.6	»	4 × 2.70	273 (23 : 61 : 16)	Gerber at Central Suspended Span
Nagaragawa	3 c 69.6	»	4 × 2.70	268 (23 : 55 : 22)	

* c = continuous
** The use ratio = The use ratio of S.M.41 (S.S.41) : S.M.50 : H.W.50
3211 tons of H.W.50 steel totally used.

Steel weight per unit floor area is shown in Figure 5 (high yield strength steel is partly used) and in Figure 6 (inclusive of bridges made of ordinary mild steel only). These figures clearly show that the steel weight has been cut down to almost half, compared with that of bridges built more than ten years ago. This remarkable result is based on the following items; wide adoption of welding techniques, use of high yield strength steels, rationalization in design methods, especially the rationalization of loading and the progress in design calculation.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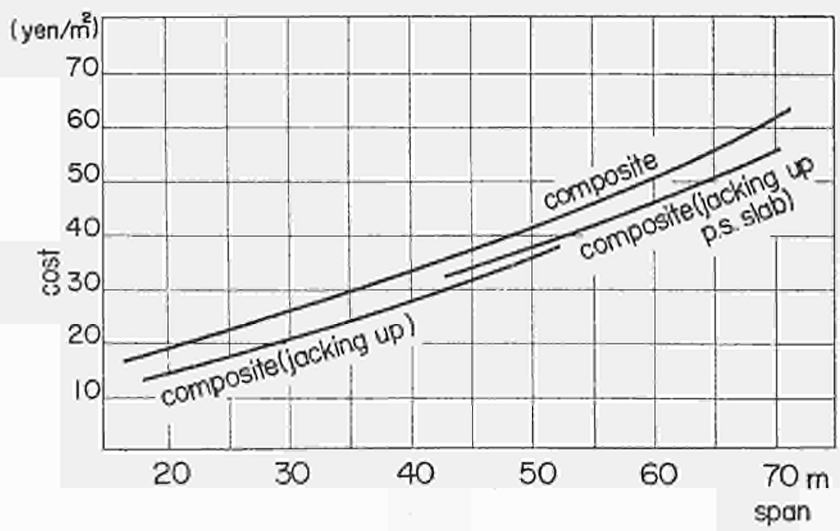
(yen/m²)
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Fig. 3

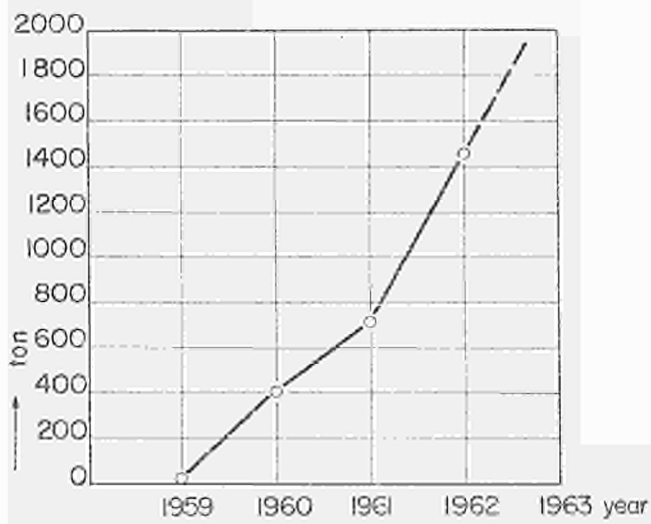
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Fig. 4

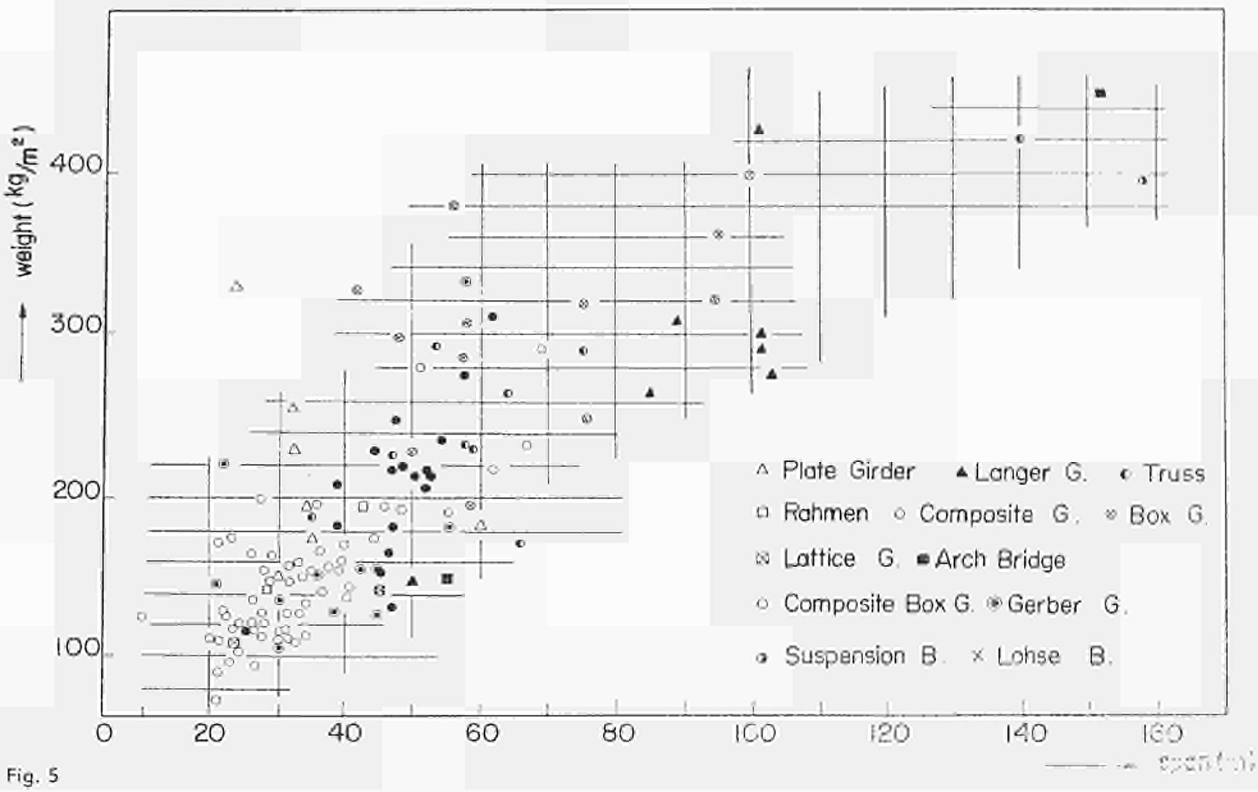


Fig. 5

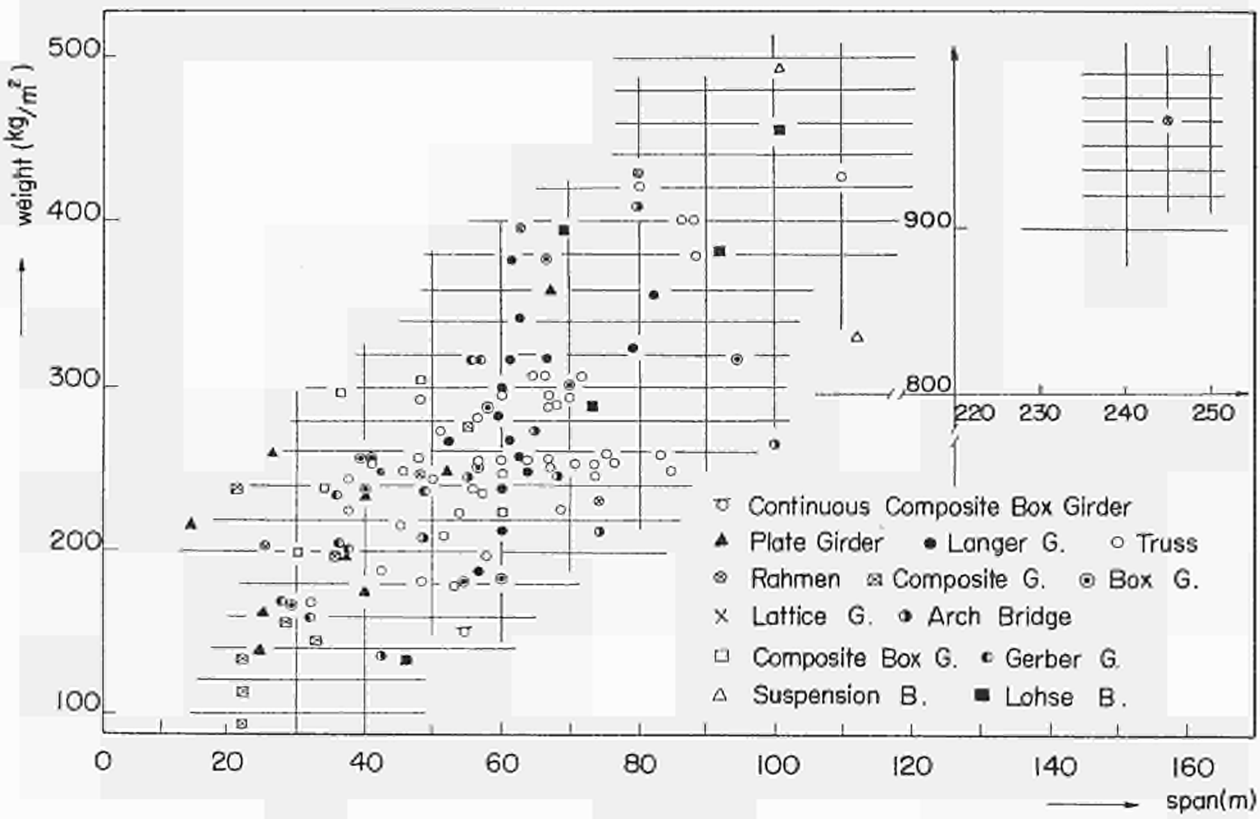
To compare the Japanese loading condition with that in foreign countries, the ratios of foreign design loading to those of Japanese loading, calculated on the standard section of the Meishin Expressway are plotted in Fig. 7. In Germany, values of design loading are rather more severe than those in Japan, because 60 to 45 ton trucks are used, while in Japan 20 ton trucks, but these values decrease as the span becomes longer. Moreover, as the loads are distributed over only two lanes (5.5m) in the case of the Meishin Express-way, the design loading values are somewhat smaller than those calculated by the Design Specification for the Steel Highway Bridges of Japan, and the value of the inside girders differs from that of the outside girders because the calculation is made as for a lattice structure.

The loading conditions in the U.S.A. are rather lighter than those used in Japan. Considering that the allowable stress in Germany is the highest and in the U.S.A. the lowest, it may be reasonably concluded that design conditions in these three countries (Germany, Japan and the U.S.A.) are almost the same, so the scantling of a bridge section and its weight may be discussed and compared with each other from the equivalent standpoint.

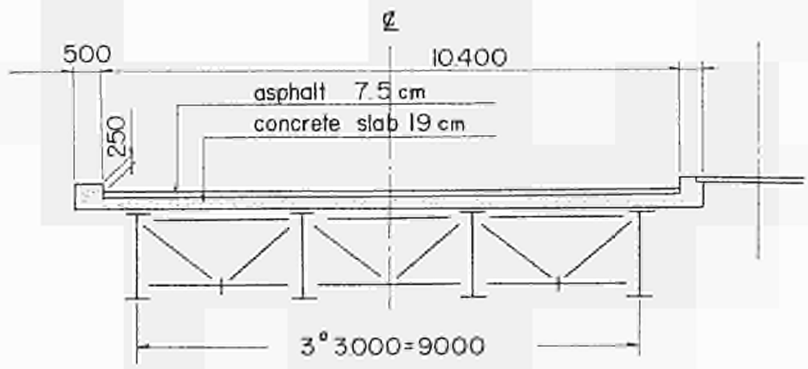
Current studies on steel bridges

As mentioned above, a lot of steel bridges have been built in Japan in recent years. It is perhaps not necessary to say that this is possible only when many fruitful experimental and theoretical studies on bridges have been carried out by researchers. As an example of recent studies in Japan, abrief introduction on research, done by the Tokyo Expressway Public Corporation will be described hereafter. In order to design and construct an elevated highway system, the Corporation was urged to conduct many experimental studies to satisfy the special circumstance that more than 40% of the total length of the highway under study consisted of elevated steel rahmen structures.

As regards rahmen structures, many studies [6,7,8] on structural types of corner connections (joining of columns and beams with various kinds of cross-section shapes) subjected to horizontal loads due to possible earth-

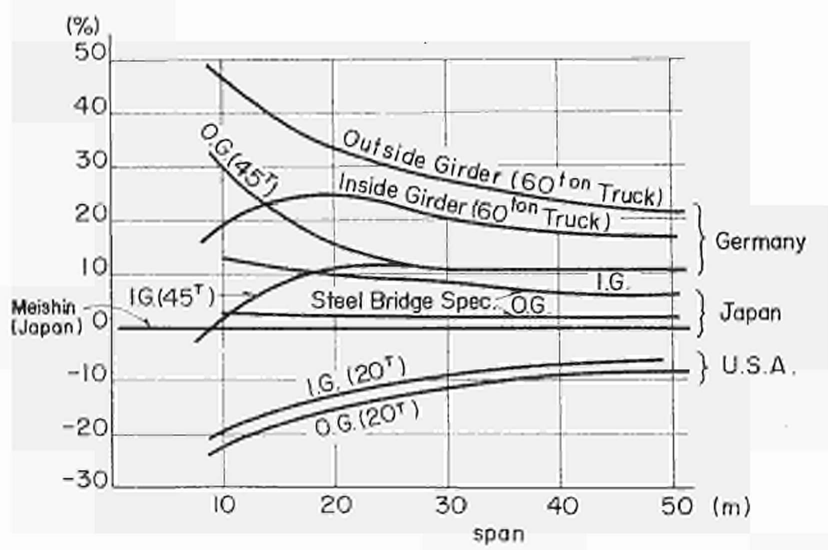


weight (kg/m²)
500
400
300
200
100
0
20
40
60
80
100
120
140
160
span(m)
900
800
220
230
240
250
Continuous Composite Box Girder
Plate Girder
Langer G.
Truss
Rahmen
Composite G.
Box G.
Lattice G.
Arch Bridge
Composite Box G.
Gerber G.
Suspension B.
Lohse B.



℄
500
10.400
asphalt 7.5 cm
concrete slab 19 cm
250
3°3000=9000

Fig. 6



(%)
50
40
30
20
10
0
-10
-20
-30
10
20
30
40
50
(m)
span
Outside Girder (60 ton Truck)
Inside Girder (60 ton Truck)
O.G.(45T)
I.G.(45T)
I.G.
Steel Bridge Spec. O.G.
Meishin (Japan)
I.G. (20T)
O.G. (20T)
Germany
Japan
U.S.A.

Fig. 7

quakes — especially important in Japan, — have been made. Corner connections of rahmen structures were considered most important. With regard to the main girders, a special structural design was sometimes required to satisfy the boundary conditions of the site. In figure 8 an example of lattice girders with varying heights is shown. This design was related to the arrangement of the center lamp and was assured by a satisfactory agreement between theory and experimental results. Figure 9 shows another example where the supporting position B is inevitably shifted to C or A owing to the arrangement of a road under the elevated road. It was found that in this case the effective width of the flange of the shallow box girder might be assumed to be fully effective over the breadth. As the curved steel bridge was often planned over roads, experimental investigations were carried out, to obtain information for the design. From the results of these large size model tests, it was found that the characteristics of stresses and deformation of the curved bridge showed a good agreement with theoretical calculation. In the case of curved girder design, the shearing strength of a girder as a thin walled closed section, subjected to torsional forces and the local buckling strength of the girder, should be well balanced.

Fig. 8

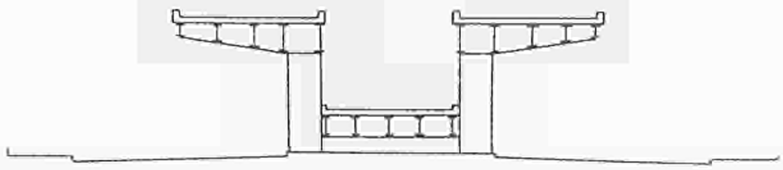
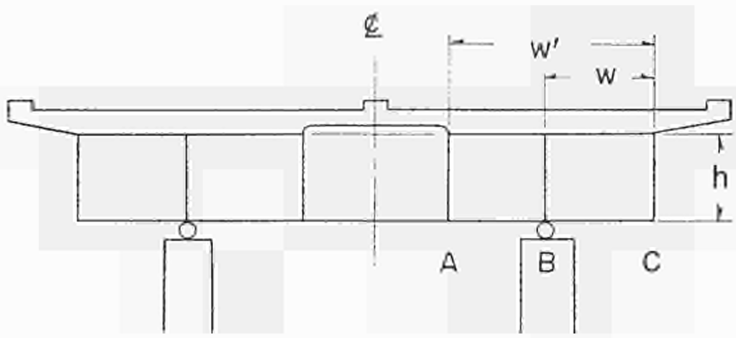


Fig. 9



Before the curved highway bridge was adopted by the Tokyo Expressway Public Corporation in 1956, the first curved highway bridge of "Shiraito Bridge" having a span of 25m. and a radius of 30m., had been erected, conform the results of the research work, including model tests of the bridge[9]. Since then, the problem on curved highway bridges has been studied and developed by many researchers and the results[10 – 22] of these investigations have contributed to the design of the curved bridge. At present, Kuranishi's[10] and Komatsu's[13] theories on the design of such a bridge are commonly accepted and it is well known that calculations based on these theories show a fair agreement with the results of actual measurements. Thus theoretical investigation along this line is going to progress by the adoption of box-section girders.

Recently, researches on suspension bridges have been carried out in Japan, resulting in the first large suspension bridge "Wakato Bridge" with a central span of 367m and a total bridge length of 680m. Fig. 10 indicates the total steel weight of the long span bridge, used both as railway- and highway bridge. When the span exceeds 500m. the suspension bridge is superior to other types of bridges. From Fig. 10, the steel weight of a suspension bridge having a central span of 1,200m is about 120,000 ton. As is well known, a 1,400m long span suspension bridge (called a dream suspension bridge) which will be hung over the Seto Inland Sea and connect Honshu and Shikoku, is under planning in Japan. To erect this long span bridge, a number of projects are now under way and it is expected that in the near future a splendid long span suspension bridge will be seen at the Seto Inland Sea.

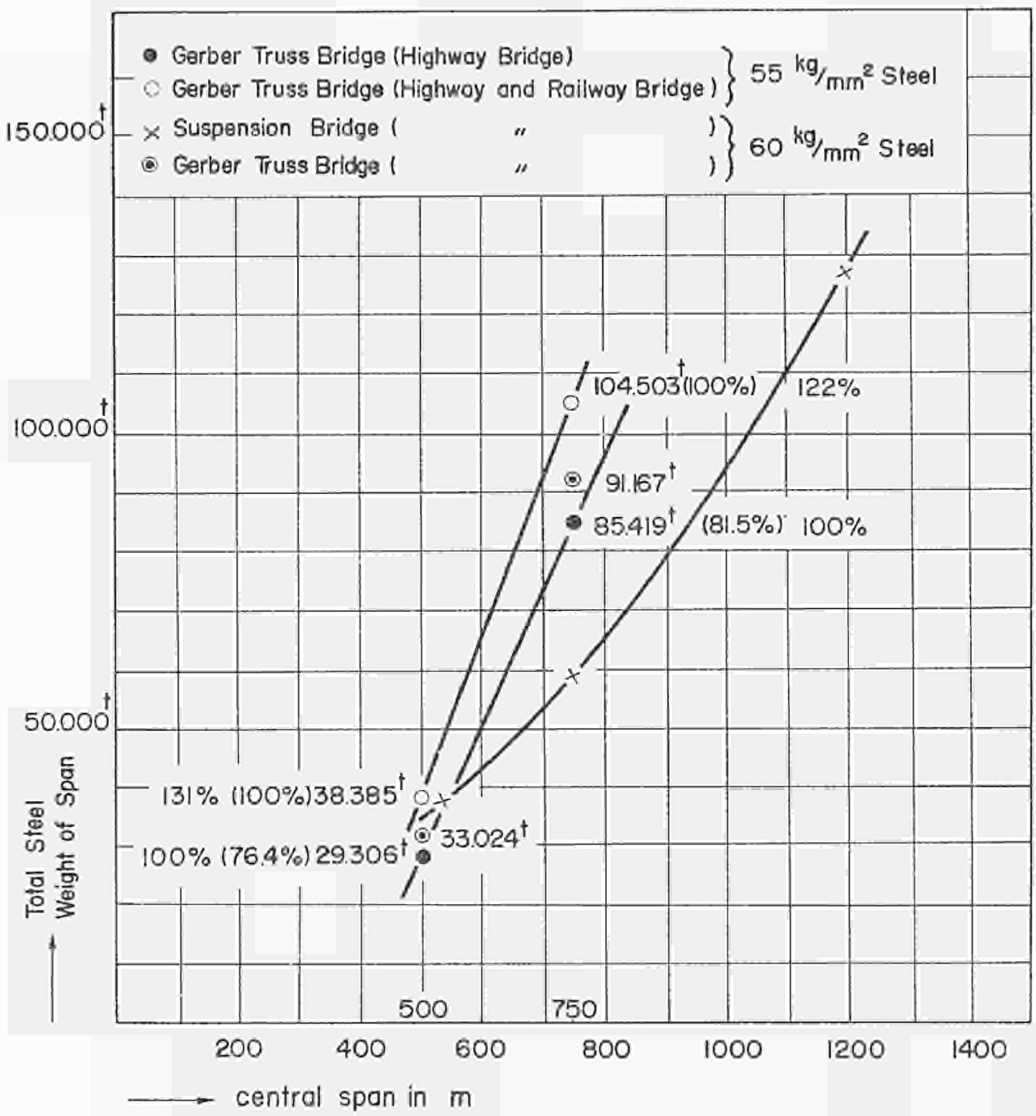


Fig. 10

Concluding remarks

Since maintenace of roads and construction of expressway systems were made part of the governmental policy, and as the budget increases, the production of steel bridges has remarkably increased and this trend will continue in the future. Besides more and more comparatively long span bridges are being constructed in the country and elevated highway bridges in the cities. As a rule, to build a steel bridge, each component part (ofter called a "block" in Japan) with about 10 tons in weight is first assembled by welding in a shop, then the blocks are transported to the site, where they are erected by riveting. Sometimes, a block weighing as much as 60 tons — in the case of the expressway bridge in Tokyo — is being shipped to the site.

Where structures are to be aligned with a road great accuracy is required. To secure this required accuracy, the welding technique has been driven to progress by researches and experience. This is well proved by paying attention to the fact that the number of technical papers on the study of steel bridges published on various proceedings and bulletins has increased by as many as half of the total number of all papers officially listed.

Moreover, together with the completion of the expressway, many pedestrian bridges crossing over the expressway have been erected. Designing these pedestrian bridges crossing over the expressway, the theory of plastic design has been applied for the purpose of weight-saving, and light gage steels and special sections have been put to practical use.

As described hitherto, it should be kept in mind that steel bridges have remarkably advanced, not only in numbers, but also in the quality of bridge structures.

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21 Akao: The experiment of the curved bridge — Bizen Bridge. Proceedings of the J.R.A., 1963.

22 Ito: On the vibration of the curved girder. Proceedings of the J.R.A., 1963.

Description of photographs

1 — Inclined framework at Honmachi.
2 - 3 — Oscillating type of pillars at Akasaka Mitzuko.
4 — Katzushimo railway-station.
5 — Prestressed concrete bridge at Dywidag.
6 — Bridge leading to highway at Hamazaki.
7 — Entry and exit of highway at Edo bridge.
8 - 9 — Junction at Nipon- and Edo bridges. (under construction).
10 — Elevated railway tracks. (Joban bridge)

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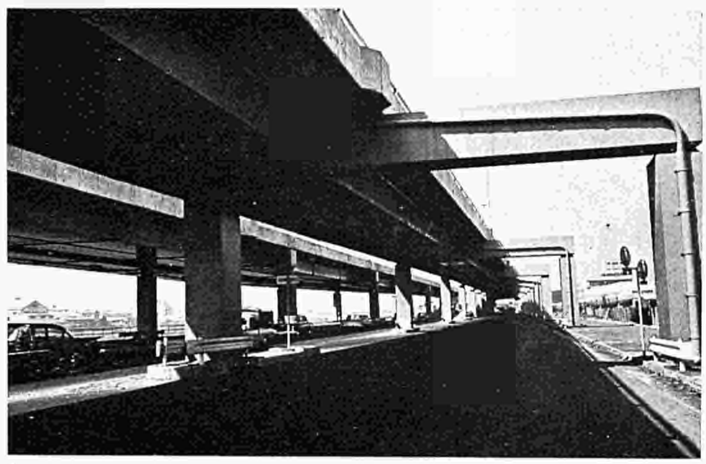

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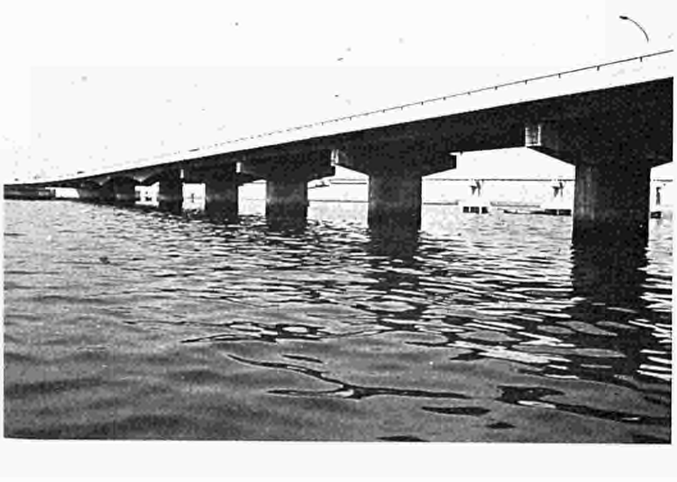

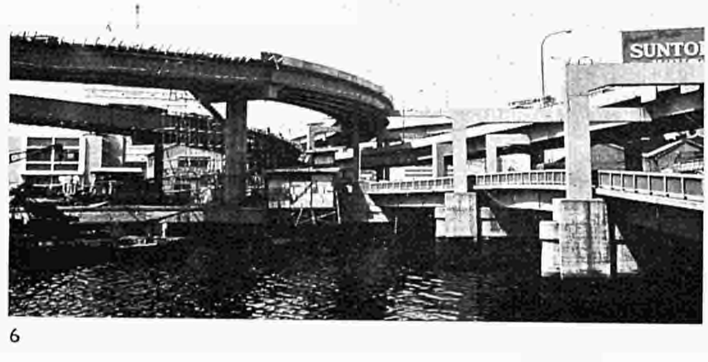

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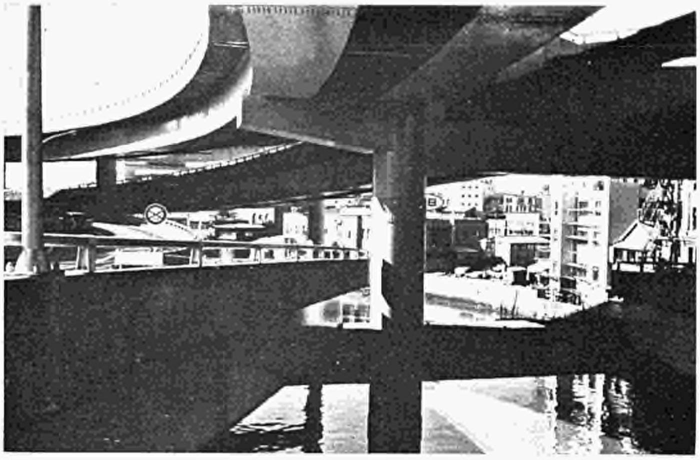
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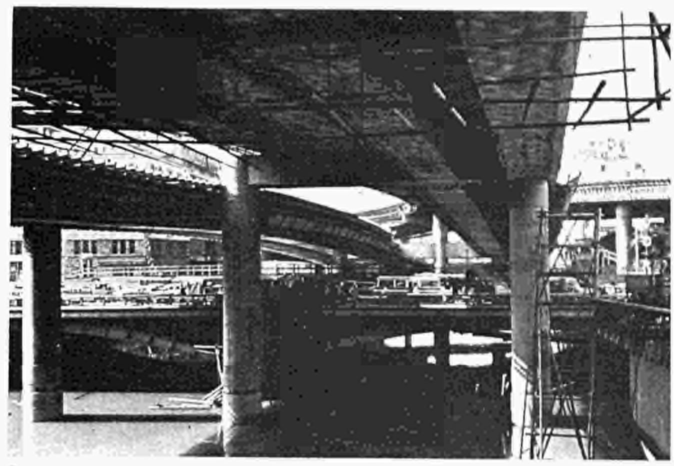
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H. SHIRLEY SMITH

Progress in the Utilization of Steel in Bridge Building

(Original text: English)

The purpose of these Introductory Notes is to refer to the recent improvements in the quality of structural steelwork and the sections in which it is available; to note improved techniques of fabrication and erection; and to show how these assets are furthering the use of steelwork in the construction of bridges — from short spans of 60 ft. upwards in motorways and flyovers to big suspension bridges with spans of 3,000 ft. or more.

Materials

The most important new development in constructional engineering in Great Britain has undoubtedly been the introduction of the new quality High Yield Stress Steel B.S.968:1962. Available not only in plates but also throughout the full range of sections, this steel is markedly superior to its predecessor, has better welding qualities and is £3 per ton cheaper. It has been largely used in the new Severn suspension bridge.

Its Yield Stress of 23 tons per square inch enables it to be employed at working stresses nearly 40% greater than mild steel. Recent comparative designs for a two-span continuous highway bridge with spans of 102 and 90 ft. showed that the use of High Yield Stress Steel saved 33% of the weight in the main girders.

The use of the new steel is facilitated by an improved standard range of sections introduced in 1962 — universal beams and columns. These are available in a large variety of weights and properties and in depths up to 36 in.

Beams of still greater depths — up to 78 in. — are now on the market as a result of a new process developed for automatically assembling and welding beams from three plates, arranged to form an I-section. These "autofab" beams are welded by submerged arc machines in the shops of the suppliers. It is possible by this process to manufacture heavy girders with equal or unequal flanges and also heavy tee sections, using any suitable quality of steel.

Turning now to the fabricating shops, significant cost reductions have been achieved by means of recent advances in techniques. These include improved quality automatic welding processes, mechanical sawing and multiple spindle drilling, and new gas-cutting machines which accurately shape steel plates to any required profile. Intensive research is proceeding amongst many other things on the production of stainless steel of structural quality.

Motorway bridges

Universal beams are ideal for spanning motorways or for carrying motorways over roads, railways and rivers. There is little fabrication required and the beams are easy to erect by mobile cranes.

In order to gain the greatest economy, the concrete road slab should be keyed to the steel beams to obtain composite action between the steel and concrete. In this way simple spans up to 90 ft. can be made, utilising 36" × 16 1/2" universal beams in high tensile steel. Utilizing a deck slab 6 1/4" thick, the total construction depth would be approximately 48" allowing for roadway camber and surfacing. A 90-ft. bridge is long enough to span across two 24-ft. wide carriageways with 10 ft. hard shoulders and an 11-ft. centre reservation, without any intermediate support.

For a dual three-lane motorway with hard shoulders, two spans of approximately 60 ft. each are sufficient, provided there is an intermediate pier in the central reservation. In this case mild steel universal beams 36" deep can be used, working compositely with a 7"-thick reinforced concrete road slab. For spans less than the maximum there is a range of universal beam sections which are suitable and other considerations such as available construction depth may determine the actual size selected.

The minimum of fabrication is required on the universal beams. Web stiffeners are necessary only over the bearings. The beams are robust to handle and when erected will carry the weight of shuttering and deck concrete.

An example is the Greengates bridge over the Leeming Bypass, which has a clear span of 88 ft., and is built of high tensile universal beams 36" × 16.5", with a composite reinforced concrete deck slab.

Viaducts and flyovers

Yet another type of steel construction was employed in the Gathurst Viaduct on the M6 which is built of continuous steel girders over the full 800-ft. length, each girder being of uniform section and of welded construction.

The approach viaducts at either end of the Forth Road bridge, which have overall lengths of 827 ft. and 1,437 ft. respectively, are a further example of economic steel design. Varying from 110 to 177 ft., the roadway spans are each supported on pairs of continuous box-girders, 2 ft. wide and 12 ft. deep, in high tensile steel. The girders were lifted into place from the ground in lengths up to 60 ft. and weights of approximately 30 tons and bolted to the preceding section by means of high strength grip bolts. The box sections were cantilevered forwards until they reached and were supported by the next pier.

On the viaduct connecting with the Chiswick flyover of the London to South Wales motorway, a length of 3,380 ft. is being built in steel which proved more economical than any other material. The curvature in plan and also three vertical curves were all embodied in the steelwork which supports a uniform composite roadway slab 7.5" thick. Four 60 ft. spans are of universal beams, the remainder being of welded plate girder construction — except for three main central latticed spans totalling 778 ft. Of the total weight of steelwork of 5,000 tons High Yield Stress Steel to B.S. 968:1962 is used for the flanges and webs of the plate girders and also in the chords of the welded box-type lattice girders. All other parts are of mild steel. The plate girders were fabricated in 60-ft. lengths, weighing approximately 15 tons each, and butt welded in situ into lengths measuring 490 ft. The maximum flange plate thickness is 3 in.

Another problem for which steel can provide the best solution is the construction of inner ring routes through built-up areas. Prof. Pier Luigi Nervi has prepared an informative series of designs in steel for single and double-deck elevated motorways suitable for use in cities. The authorities at Coventry made use of the peculiar versatility of steel in the design and construction of the Moat Street flyover junction to cross a ground level roundabout. The dual carriageway of the inner ring road is carried over the junction by means of a 735 ft. long steel viaduct. Structural steel was selected by the city engineer after detailed investigation of alternative methods had shown that steel afforded both the lowest cost (including an allowance for maintenance) and the greatest speed of construction. In the event, the 820 tons of steelwork was erected in only six weeks.

Such a speed is not exceptional for steel erection and even more impressive rates can be quoted. The Birmingham Carbridge, which is 790 ft. long, was put up in less than 30 hours during one weekend at a busy traffic junction. Similarly more than 250 tons of universal sections were erected in six hours to build the Shawhead flyover between Glasgow end Edinburgh.

Cable-braced bridges

The modern tendency in bridge design is increasingly towards plate and box girders in preference to trusses. Of recent years in Great Britain only a few of the longest spans, such as the Widnes-Runcorn arch and the stiffening girders of the Forth and Tamar suspension bridges have trussed steelwork, amounting to perhaps 20% of the total bridgework fabricated. Whereas nearly all small spans, such as those at Barton, Thelwall, Neath and Maidenhead, and recently even bigger spans such as the Usk and Wye bridges, culminating in the deck of the huge Severn suspension bridge, are of the plate or box girder type.

Following on the first modern cable-braced or "bridle chord" bridge of 600-ft. span built over the Strömsund in Central Sweden in 1956, this type of construction has found increasing favour. In 1961 the first mono-cable bridge — in which a single main supporting system, on the longitudinal centre line, replaced the conventional dual system — was built over the Norderelbe at Hamburg. The first bridle chord bridges with dual- and mono-cables respectively in the U.K. are the 500 ft. span over the Usk at Newport, Mon., opened in 1963, and the 770 ft. span now under construction over the river Wye, close to the Severn bridge.

Further striking economies in weight and cost are rendered possible by the bridle chord design. The chief structural element on the Wye bridge is the trapezoïdal box girder, chosen for its great torsional strength and rigidity. This form of deck was developed, as we shall see, for use on the Severn suspension bridge. The main span of the Wye bridge is flanked by side spans of 285 ft. At each end of the main span a single box section tower 96 ft. high, hinged at road level, supports the central staying cable which is anchored to the box girders 255 ft. either side of the tower. The advantage of a mono cable is that its load is not affected by unsymmetrical live load on the bridge, which is resisted by the great torsional strength and rigidity of the closed box section. This results in a cable economy of some 70% as compared with a dual cable system.

The viaduct portion of the Wye bridge consists of two continuous steel box girders, varying from 182 to 210 ft. in span, supported on simple box section trestle legs. As much fabrication as possible is done off site and all final connections will be welded in situ by means of portable automatic or semi-automatic equipment. The total weight of steel in the entire project is 8,000 tons, half of which is weldable high tensile and the other half mild steel.

The Forth and Severn bridges

The Forth Road bridge with its main span of 3,300 ft. was opened in september 1964. Although it is capable of carrying the heaviest traffic, including vehicles weighing 250 tons each, this bridge is lighter in weight and more economical in cost than any bridge of comparable span in the world. The lightness in weight of the bridge, indeed, combined with the fact that the site is in latitude 56° North — much further north than any other major suspension bridge, and notorious for its high winds and gales — presented novel problems during erection in the avoidance of aerodynamic oscillations.

The tower legs are each made up of welded high tensile steel box sections, 47 ft long, plated together so that each leg is finally divided into five cells. The ends of the boxes are machined to butt together and have internal flanges connected by vertical high strength bolts to take the stress during erection. This construction is simple to erect and provides a flush external surface which facilitates protection and gives a neat appearance. Much saving in weight results through a greater concentration of material at the outside surface, compared with the conventional form of tower in which the legs are divided into many cells each 3′6″ square.

On the main span, each ton of dead load requires half a ton of wire in the main cables to support it. It therefore proved economic to use a stiffened steel battledeck in the main span which, though more expensive in itself than a reinforced concrete slab, was much lighter and therefore effected an overall economy. In the stiffening trusses, hollow completely sealed steel box members were used wherever practicable and throughout the design care was taken to ensure that all parts requiring painting were readily accessible. Moreover, the initial protection of the steelwork by means of grit blasting, metal spraying and four coats of paint was the best that could be devised. Though costly, this treatment should obviate the necessity for repainting except at infrequent intervals.

The final design of the Severn bridge was made a few years after that of the Forth Road bridge. Since the Tacoma Narrows failure in 1940, suspension bridges have all had deep latticed stiffening girders incorporated in the deck to eliminate torsional oscillations due to aerodynamic action. Gilbert Roberts has explained how, on behalf of the Consulting Engineers, he proposed a series of tests which was carried out by the Aerodynamic Division of the National Physical Laboratory on models of very much shallower decks, including a plated box girder. This was enclosed by stiffened plates at the top and bottom and inclined plates at the sides, so as to make it in a large measure streamlined. By this means a satisfactory torsion shell box or aerofoil was finally evolved which extends throughout the span and is suspended from the cables by inclined. hangers. The upper surface of this aerofoil box, which is only 10 ft. deep, constitutes the deck on which the roadway is constructed, and the projecting platforms at the sides carry the cycle tracks and footways. The purpose of the triangulated hangers is to damp out aerodynamic oscillations which might otherwise occur with winds blowing slightly upwards from the horizontal. The advantages of this design lie not only in the economy of the deck itself, but also in the fact that its shallow aerofoil profile reduces the wind loads on the bridge and thus effects a saving of material in the cables and the towers as well.

The legs of the 400-ft. high towers are not cellular, but each consists of a single box, built of four stiffened steel plates, shop-welded and site-bolted. They are braced together by deep steel portals. Although the span of the Severn bridge is only 60 ft. less than that of the Forth Road bridge, the above measures have effected a very considerable saving in the weight of steelwork and the overall cost.

In a recent issue of the American journal "Engineering News-Record" the following comments appeared comparing British and American costs on suspension bridges being built and projected:

"Some cost figures on Scotland's Firth of Forth Road Bridge, Europe's greatest to date: this structure cost $30.6 million, it has a main span of 3,300 ft. Some cost figures on this bridge need consideration: its structural steel cost $120 per (U.S.) ton. Fabricated and delivered, this steel cost $298 per ton. Erection cost another $300 per ton. Superstructure miscellany, such as hand rails, paving and lighting cost another $50 per ton of superstructure steel. Foundations cost $187 per ton of superstructure steel. And land, toll equipment and engineering lump together as $25 per ton of bridge steel. Total: $860 spent for each ton of bridge steel used. And welding instead of riveting cuts steel tonnage at least 10 per cent.

Then, the British designers of this bridge did some „value engineering" and came up with a new suspension bridge — the Severn Bridge, now under construction — only 60 ft. shorter in its main span than the Forth Road Bridge, but 35 per cent lighter and costing almost $9 million less. They drastically reduced the steel tonnage required by going to a suspended box only 10 ft. deep instead of deep trusses. And they further cut the cost per ton of fabricated steel from $298 to $263 in the process, despite the increase in costs for labour and materials.

Next, these same British designers designed the proposed Humber River Bridge with a 4,580 ft. main span, 320 ft. longer than the world's longest. Although its record main span would be 1,340 ft. longer than that of the Severn Bridge, the Humber is estimated to cost only $2.8 million more between anchorages than the Severn. The proposed Humber crossing would have a total cost, including engineering and approaches, of $35 million.

The Narrows Bridge in New York will be finished late this year with its 4,260 ft. main span, the world's longest. And its $325-million cost ($240 million between anchorages) will set a world's cost record that should stand for years to come.

The great discrepancies in cost between Britain's bridges and our U.S. world-beater need more explanation than the Scottish wage rate $1.05 an hour in the shop and $2.10 in the field. Actually the costs per ton in place do not differ so much between the U.S. and Britain. The big difference is in the number of tons. The British figure the Narrows Bridge uses eight times as much steel as the longer-span Humber design calls for. This works out to 4.5 times as much per square foot of roadway, a smaller factor because the Narrows Bridge will have two decks.

Then there is the $100-million Delaware Memorial "twin", now under construction alongside the original built 13 years ago for $40 million.

Are the British becoming too daring in suspension bridge design? Did the Narrows Bridge cost 4.5 times what it might have? Is some value engineering in order in the U.S. on suspension bridges?"

In conclusion it is good to see that the art of bridge building has never been more dynamic, the materials available for use more diverse, or the forms in which they are used more imaginative than they are today. But, designers must be watchful at all times to resist "the fascination of what's difficult." The merit of a good design will always lie in its simplicity, with much thought given to ease and economy of erection. To the same end, scantlings should not be reduced to the bare minimum unless there is ample background of experience to ensure that they can resist not only the calculated static loads, but also the imponderable dynamic forces they may have to withstand both during erection and under working conditions. Otherwise the saving in cost of material may be many times outweighed by the extra costs incurred in time and labour on the site.

Description of photographs

1 — Midland link bridge — Walsall, England.
750 tons of 36" Universal beams spanning 89 feet.

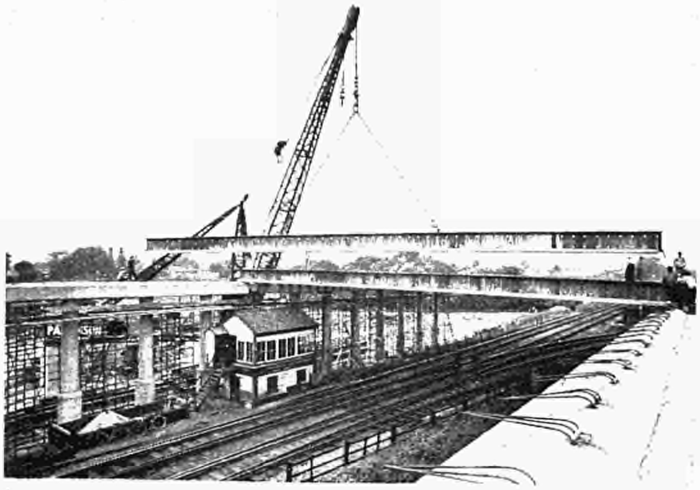
2 — Shawhead flyover bridge.
Universal beams manufactured at the Lackenby Beam Mill of Dorman Long (Steel) Ltd. were used in the construction of this fly-over bridge on the main Edinburgh-Glasgow road. 256 tons of structural steel were delivered and erected in one day with the use of mobile cranes.

3 — Moat Street flyover, Coventry.
View from behind the West Abutment showing the curved section. The Moat Street Flyover, which forms part of Stage III of the Coventry Inner Ring Road, incorporates 820 tons of steelwork consisting of box plate girders and columns, supplied and erected by Dorman Long (Bridge and Engineering) Ltd., Middlesbrough. The main contractors are G.R. Yeomans Ltd., of Ryton-on-Dunsmore and the work is being carried out to the design and under the supervision of Mr. Granville Berry, M.I.C.E., M.I. Mun. E., City of Coventry Engineer and Surveyor.

4 — Severn Bridge in course of erection.
Foundations, Towers & Cables completed - August 1964
Consulting Engineers - Mott, Hay & Anderson, and
Freeman, Fox & Partners
Architect - Sir Percy Thomas, P.P.R.I.B.A.
Contractor for substructure - John Howard & Co. Ltd.
Contractor for superstructure - Associated Bridge Builders Ltd.

5 — Proposed Humber Bridge - Artist's Impression.
The span of this bridge would be the longest in the world.
Details of the span and cost are given in the paper.

6 - 7 — Components of elevated road.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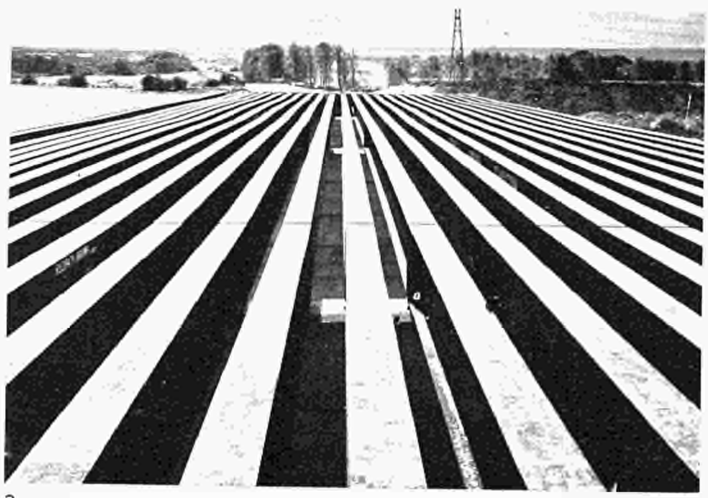
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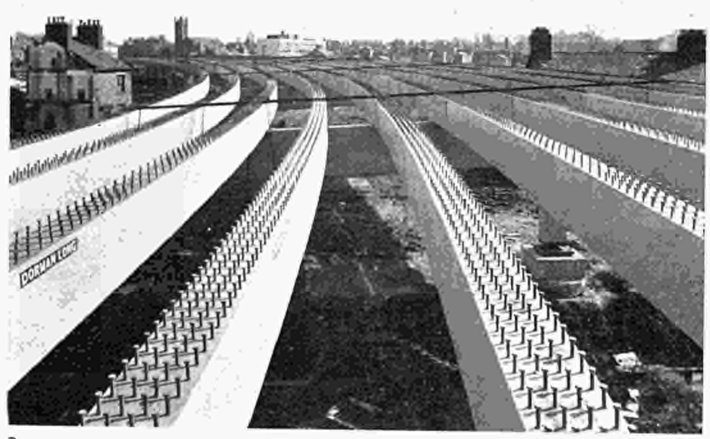

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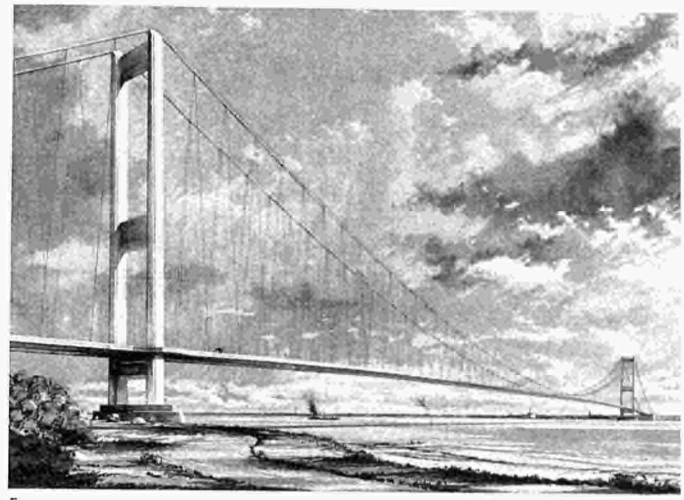
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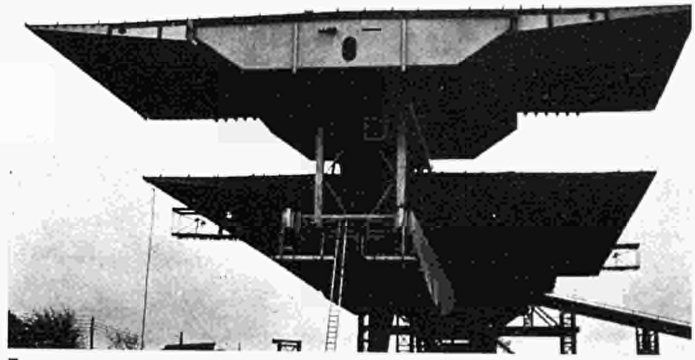
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Hellmut HOMBERG

Recent Developments in German Bridge Building

(Original text: German)

General

Since the end of the war we in Germany have had the job of rebuilding a great number of bridges which were destroyed during the war, and this job had to be carried out with the most economical use of existing funds. To effect this economy, bridge engineers are turning to new types of structures and to new design methods.

Statically more accurate evaluations of stresses in grids

The starting point for all statical investigations is the manner of interaction of all the members of a bridge girder system. Up to a few decades ago the structural elements of a bridge were designed and constructed as an independent girder system in the vertical plane. More thorough means of calculation have to be used if the effect of the grid-like cooperation of all the members in the horizontal plane is to be evaluated. As a result of their investigations the authors [1, 2, 3, 4] have developed methods of calculation of grids, which now enable us to evaluate, for any beam grillage or grid, not only the rigid cross-connections, the torsional stiffness of the main grillage beams, but also the shear stiffness of the deck. Similarly, attention was directed to the exact analysis according to plate theory, of the stress conditions in the deck plates of roadway bridges. This first became possible using the tables by Bittner [5] and Pucher [6] which contained much numerical data on the influence surfaces of isotropic plates of constant thickness.

Somewhat parallel to the above-mentioned research into isotropic plates ran the development by the German steel industry of lightweight steel roadways, a development which opened up an extensive new field for steel. These lightweight steel roadways were built up as closely meshed grillages and were designed on the orthotropic plate, or grid theory. The first material to be published on this subject in Germany originated from Cornelius [7] and from the authors [1 4].

Composite action steel bridges with roadway deck slabs

In all calculations on the plate theory basis, it is assumed that the plate elements are homogeneous and of constant thickness. In fact the deck slabs of our composite action steel bridges have been, and are being, built generally with haunches, and with the introduction of pre-stressed concrete for the deck units the deck-spans have increased considerably. The question ought therefore to be whether these days it is good enough to design deck slabs of composite action steel bridges according to the uniform thickness isotropic plate theory. To answer this question the author and his assistant Mr. Ropers, aided by a research committee set up by the Federal Ministry of Transport have carried out exhaustive tests into the behaviour of slabs of variable thickness with the object of producing influence surfaces for such slabs. The first results of our experiments were published in 1963 [8] and covered cantilever slabs of varying thickness; they show that considerable errors can arise where the design is based on the theory of uniform slabs.

The author has therefore carried out a considerable amount of research in order to obtain, besides those methods of calculation for isotropic slabs already published, solutions for isotropic slabs of variable thickness. Much use was made of electronic means of calculation.

Figure 1 shows the representation of the contour lines for a slab simply supported on two line bearings with two cantilever arms. The three bays are of varying thickness, the thickness of the cantilever slabs increasing linearly to the bearing line while a parabolic shape is adopted from the bearing lines to the middle. The influence surface shown is for the slab moment m_{1y} and gives the moments parallel to the free edge under a wandering unit point load. Until now, influence surfaces for such a continuous system have not been available even for uniform thickness isotropic slabs.

Figure 2 shows the influence line for the moment m_{2x} normal to the support line with the point under consideration over the bearing line itself.

Figure 3 shows the influence surface for the moment m_{1x} at the fifth point of the centre panel. This is certainly quite interesting in that both positive and negative values are produced between the bearing lines.

In order to carry out a thorough design of wide span deck slabs, we have tried to put our new found knowledge to use in a practical, useful and competitive type of bridge. It appeared that the above-mentioned slab section, carried on two line bearings parallel to the bridge axis, could still be very economical in the case of quite large spans. We have worked out designs for bridges of "autobahn" width in which the overall width of 31.25 m. (102'6") is carried by only two steel main girders.

The bridge shown in figure 4 is to span a valley; it has four spans of 65 m (213'3") each, the breadth between parapets being 31.25 m (102'6"). The two main steel girders are overlaid by a pre-stressed concrete slab (Fig. 5) of span 16.50 m. (54'1 1/2") with 7.50 m. (24'7 1/4") cantilever arms. In spite of the large span of the deck slab and its considerable thickness, the bridge can be built very economically if one takes into account the fact that the cost of steel structure can be reduced to a hitherto unknown minimum.

Attention should be drawn to the fact that the steel is disposed essentially only in the two main girders, which are economically proportioned structurally. Plated sections, gen-

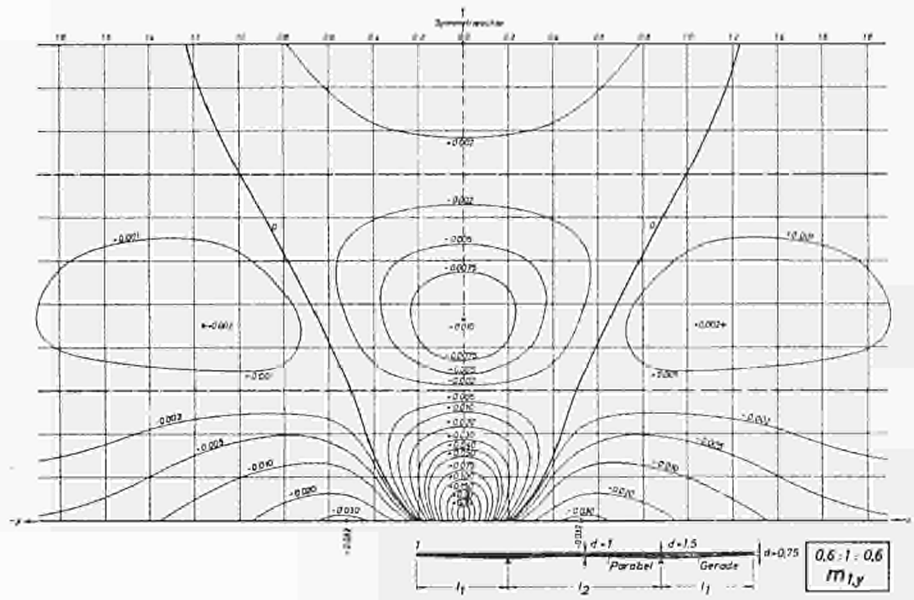

Fig. 1

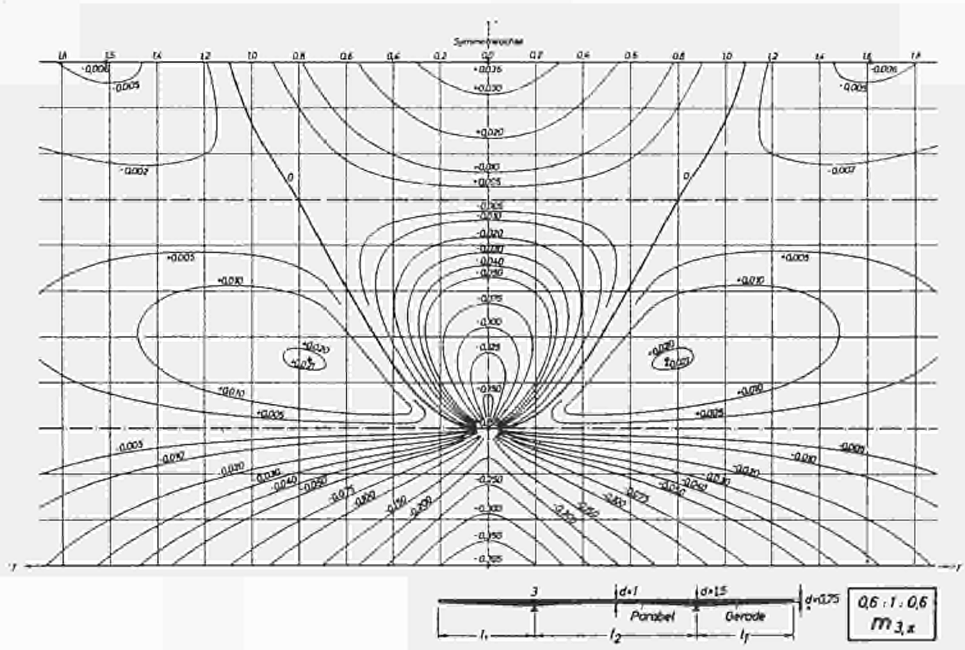

Fig. 2

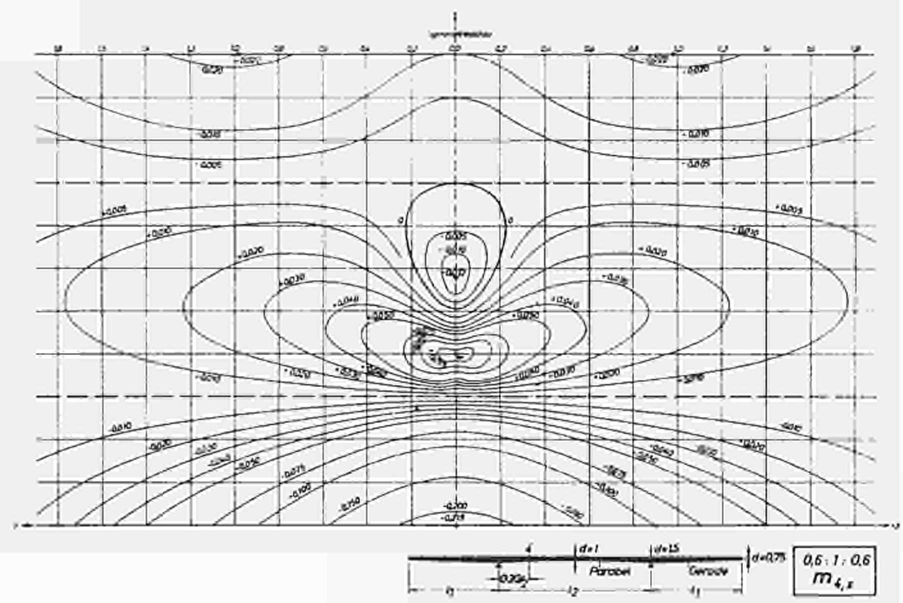

Fig. 3

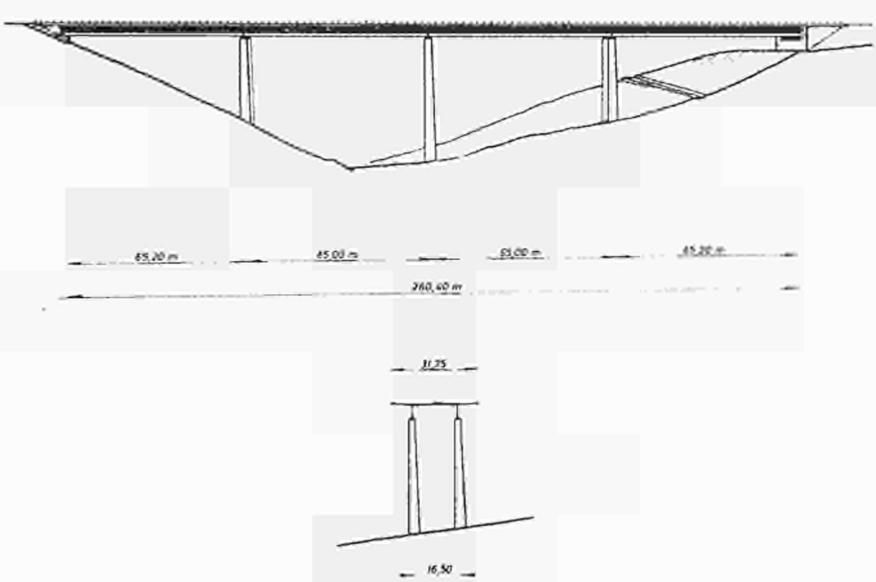

Fig. 4

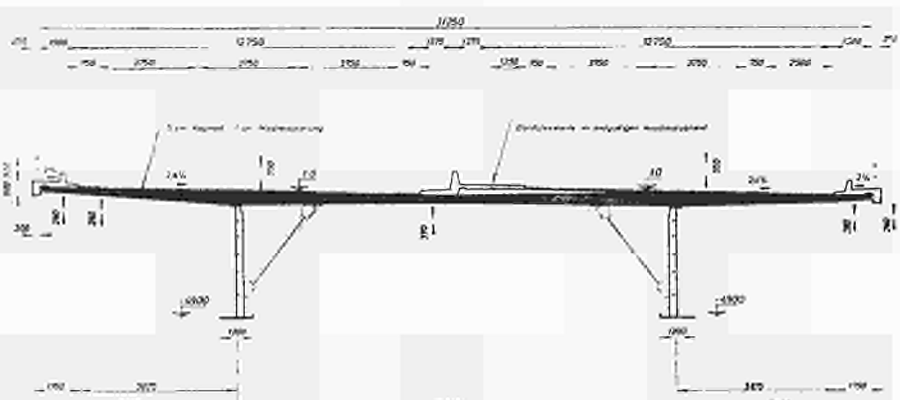

Fig. 5

erally with narrow cross girders in composite action bridges are wasteful.

Because the slab has a constant cross section along the length of the bridge it can be concreted in appropriate stages, and in this way expenditure on shuttering can be reduced in comparison with the previous methods. The bracings used for supporting the main girders against each other during erection are placed under the space allowed for the shuttering; the diagonals for the stabilisation of the bottom flange are not erected until the deck slab has been concreted.

I believe that in the future steel bridges will generally be designed in this way. They are even more competitive with reinforced concrete bridges. The fact, as shown, that the construction makes use of a minimum number of structural parts, helps to keep down the cost.

Cable-stiffened bridges

The bridge systems mentioned above are in general most appropriate for sites requiring medium size spans with an upper limit of about 100 m. (330'). Beyond that span, steel bridges come into another wide field of application in the bridging of larger spans, as are especially necessary over waterways. In the last few years the cable stiffened bridge has been developed, especially in Germany, to cover these larger spans, and bridges built to this sort of system have spans from 172 m. to 280 m. (574'3" - 918'7").

Side by side with the development of cable-stiffened bridges, a type of construction proposed by Haupt[9] is coming into use; this can be described as a spine girder bridge, in which the main structural form and also the cable-stays are concentrated along the centre line of the bridge. Off-centre loading on such a structure is carried in a combined manner; the vertical loads being carried on the spine while the torsional moments resulting from the eccentricity of the loading are carried on a torsionally stiff hollow box stiffening girder.

Figure 6 shows the Rhine bridge carrying the "autobahn" between Cologne and Leverkusen — the largest cable stiffened spine girder bridge yet built; the erection of the steel structure was finished some weeks ago.

Fig. 6

Figure 7 shows the general arrangement of the bridge with its centre span of 280 m (918'7"), side spans of 106.26 m (348'7 ½") each and land approach spans of 97.40 m. (319'6 ½") each.

The pylon and the cross section of the bridge are shown in figure 8, and the structural format of the spine girder bridge which has a slender hollow box girder with wide cantilever arms is clearly seen. The design of the bridge, which was carried out in my office, allowed a considerable margin to cater for the large secondary longitudinal stresses in the box due to torsion in the section near to the middle supports. We realized that design on the basis of the till then valid theory of secondary torsional forces, while taking into account the shear deformation from St. Venant's torsion, nevertheless does not include the deformation from the secondary shear stresses. Meanwhile a publication by the late Professor Heilig[10] points out that the maximum values of the longitudinal stresses ascertained according to the theory of secondary torsional forces do not occur but are reduced to a fraction of their previous values. Heilig succeeded it including the deformations due to the secondary shear stresses in his investigation of secondary torsional stresses of box girders. In addition to Heilig's work it must be said that the elasticity of the cross members, which in contrast to their theoretical condition do allow appreciable deformation of the cross sectional shape, further reduces the longitudinal secondary stresses.

During the development and design of the Rhine Bridge at Leverkusen it became evident that the introduction of very large concentrated cable loads into the stiffenning girder was leading to extraordinary difficulties as a maximum load of 5,500 t. can develop in the upper of the two cables, and considerable stiffening of the web plates, deck plate, and lower plate was necessary in order to distribute the load from the cable's narrow anchorage point on the centre line of the bridge out into the full width and depth of the box section of the stiffening girder. In view of the cramped conditions for welding work on the plates, this might represent the limit of current technical capabilities and for this reason impose an upper limit for the application of cable stiffened bridges.

The facts arising from theoretical investigation into the distribution of loads in plates of widely varying thickness were not at our disposal when the design was started and therefore, in order to obtain some knowledge about the load distribution in the cable attachment zone, an approximate calculation on the lines of Ebner-shear-field theory[11], as used in the aircraft industry, was carried out.

Figure 9 shows the statical system which we have taken as a basis for the calculation of stress conditions in the load introduction zone. To start with, for clearer presentation and for simplification of calculation, the upper deck plate and the lower plate of the box section are transformed into the plane of the web plates as idealized web plates. The left hand side of the diagram shows which areas of the resultant plate represent the web, upper flange and lower flange. Next the continuous functions, which occur in the plate as longitudinal and shear transferences, are divided up into

Fig. 7

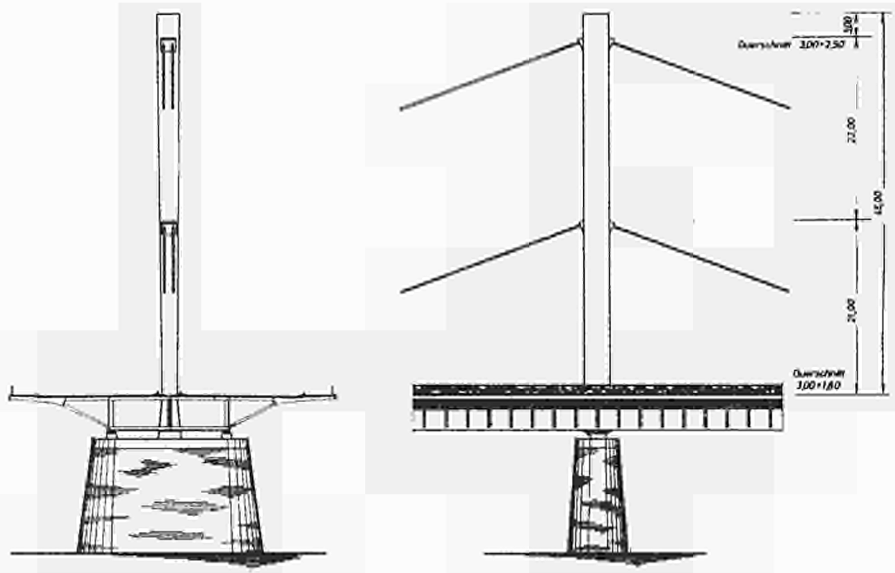

Fig. 8

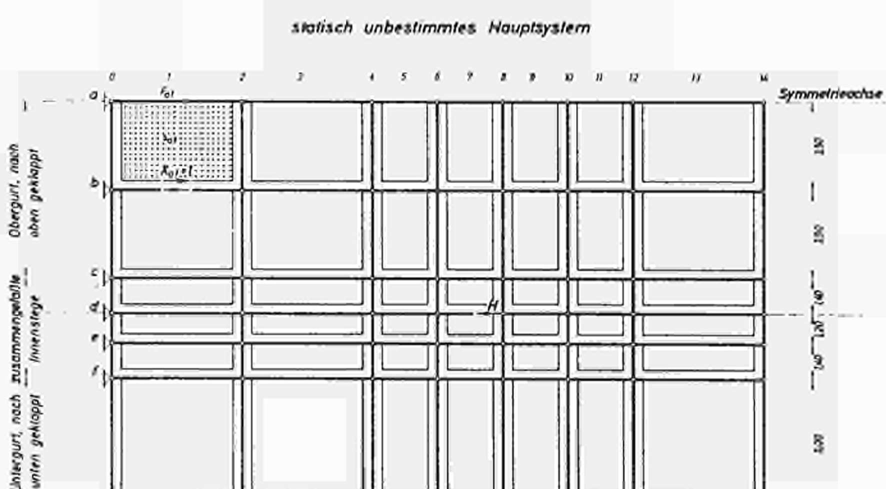

Fig. 9

individual discontinuous functions of equal value. The solid lines represent the individual longitudinal and transverse members which correspond to the longitudinal and transverse elongation stiffnesses respectively of the plate in the actual zone. In between there are individual shear membranes which represent the shear stiffness of the actual plate system, the value of the elongation stiffness from panel point to panel point, the shear stiffness of the individual shear systems being taken as constant in any section. Conversely the elongation stiffness of the individual members and the shear stiffness of the individual shear systems show considerable mutual variation. The apportioning of the acting unit force H to the individual longitudinal and transverse members is done as a statically indeterminate calculation.

The shear forces determined in this system are shown in figure 10, which illustrates the large increase in the magnitude of the shear forces in the area of the application of the load. Otherwise it is seen that because of the asymmetry of the systems in the vertical direction, the shear flow is also considerably asymmetric to the attachment point. On the other hand symmetry exists in the direction of the acting force in front of and behind the attachment point. The step-like representation of the shear flows shown in the diagram is the direct result of calculation on the discontinuous system basis. In fact the shear flow has to be introduced not in the step-like manner but as a system with a constantly developing surface whose mean ordinate at each shear field corresponds to that of the stepped system.

Figure 11 shows the axial forces found in the individual longitudinal members. The outermost members a and g have especially large normal forces, because they include the proportionally large areas of the upper and lower flanges respectively.

Figure 12 shows not the member loads but the stresses, and as already shown in the figure relating to isotropic plates of constant thickness it can be seen that the normal stresses decrease from inside to outside.

Finally Figure 13 shows the normal forces in the cross members of the plate; the magnitude of these forces necessitating careful design and proportioning.

Using these results shown it was possible to carry out a suitable design of the cable load introduction zones of the plate, and on this basis to arrive at the actual plate thicknesses of 32 mm. (1 1/4") for the deck and 35 mm. (1 3/8") for the 2 webs; elsewhere the normal thicknesses were 12 mm. (1/2") and 10 mm. (3/8") respectively.

These investigations show that the introduction of considerably larger cable forces into the stiffening girder is not practicable with existing technical resources, and a definite limit is therefore set for the use of such cable stiffened bridges. In addition the load introduction areas in bridges already built are certainly expensive on account of the welded strengthening.

These considerations led the author, to propose for the Bonn-North Rhine Bridge (Figure 14), a cable stiffened system which avoids heavy single cable and which continuously supports the stiffening girder for long lengths. The bridge, the cross section of which largely resembles that of the abovementioned Leverkusen bridge, has 20 cables lying on top of one another on each pylon. These cables no longer consist of bundles of strands as at Leverkusen, but of prefabricated bundles of strands as at Leverkusen, but of prefabricated single cables. The principle of the new system is the introduction of the cable loads continuously into the stiffening girder thus obviating reinforcement of the deck. This system also allows the cable anchorages to be simplified because prefabricated cables which can be easily attached are used in the construction.

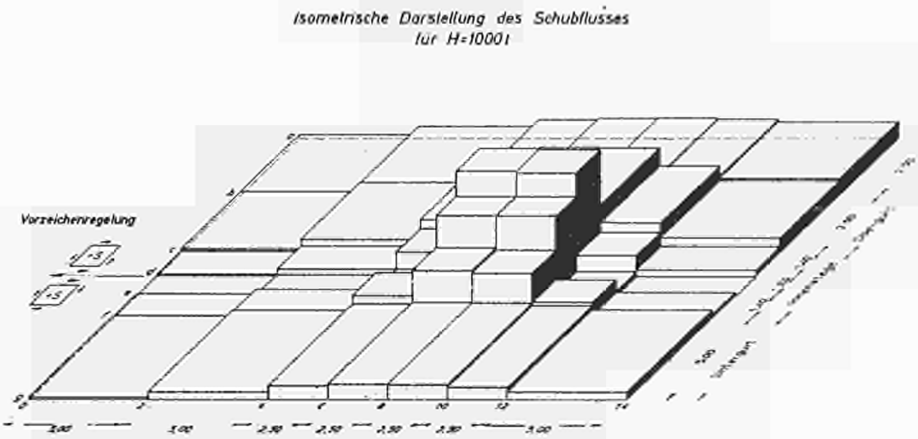

Fig. 10

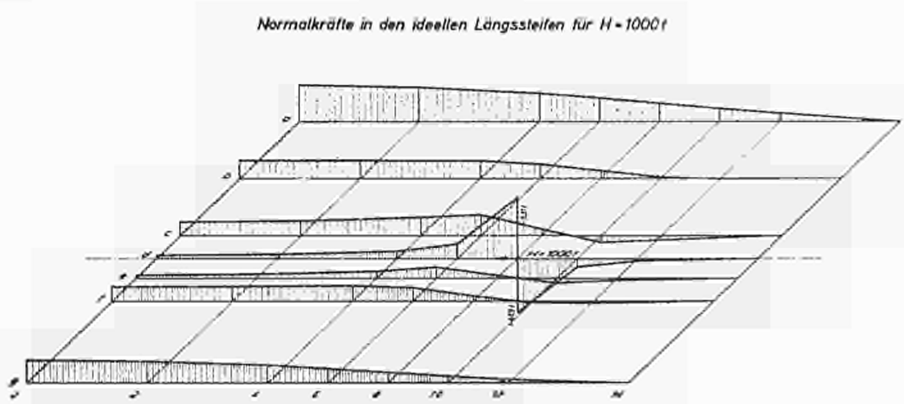

Fig. 11

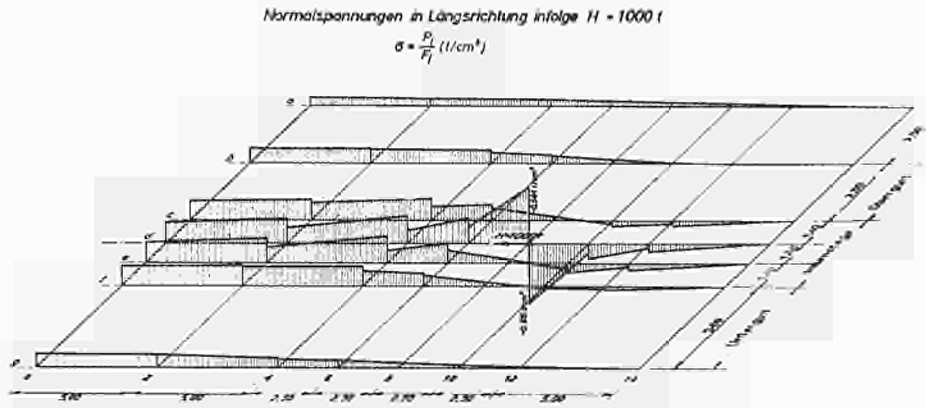

Fig. 12

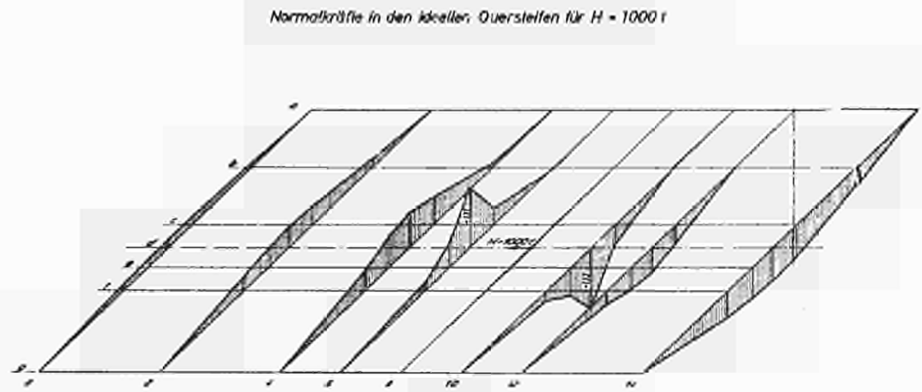

Fig. 13

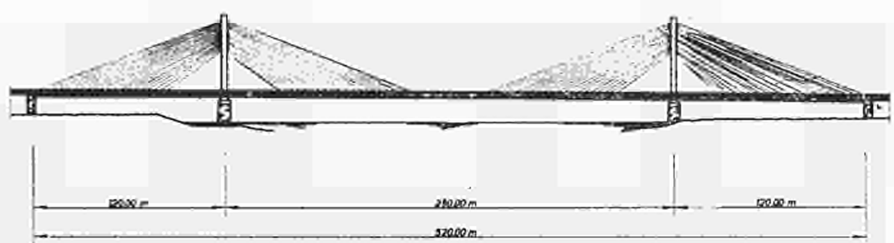

Fig. 14

Proposal for a bridge over the Channel

This new type of cable hung bridge or as I call them "Seilwerk" bridges (literally: cablework bridges) as well as being able to span considerably larger distances than hitherto, can be used in cantilever form. By using comprehensive statical and economic comparisons I could prove that these bridges show an advantage over suspension bridges for spans of 500 m. (1640') to 800 m. (2620'). Moreover there is no difficulty in having a number of "Seilwerk" bridges "end to end", in order to provide thoroughfares for large shipping lanes.

We know that the Governments concerned are considering plans for a permanent traffic link between England and France. The designs put forward so far by the steel industry for the bridging of the Channel provide for a few large shipping openings only and have thus incurred the opposition of all parties interested in Channel traffic. For this reason the Anglo-French committee responsible for planning the scheme turned to the tunnel proposals.

The above-mentioned system of the "Seilwerk" bridge makes it possible to bridge the shipping lanes with large spans, and the following figures show a few details of my proposal for the Channel bridge. These were sent on by me to the English and French Governments some months ago.

Figure 15 gives an overall view of the bridge scheme which would be about 35 km. (22 miles) long. Close to the English coast there are 25 shipping lane openings each of 500 m. (1640') spanned by „Seilwerk" bridges; on the French side there are only 3 spans each of 500 m clear opening since the traffic is not as heavy as that close to the English coast on account of the shallows.

Figure 16 shows an enlarged section of the "Seilwerk" system over the large shipping openings; it will be noted how the bridge scheme is made up of a number of identical elements. A section from midspan to midspan is complete in itself and can be erected independently of other sections and arranged in line as convenient. Longitudinal changes in the length of the bridges due to temperature or loading variations can be taken up in expansion joints situated at the centres of the spans.

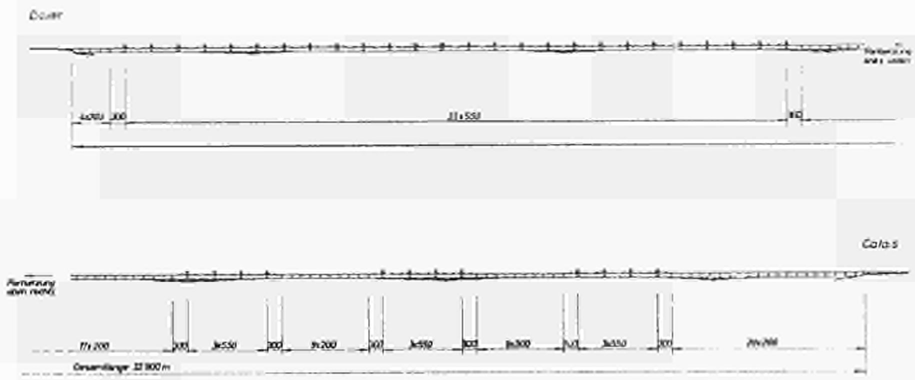

Fig. 15

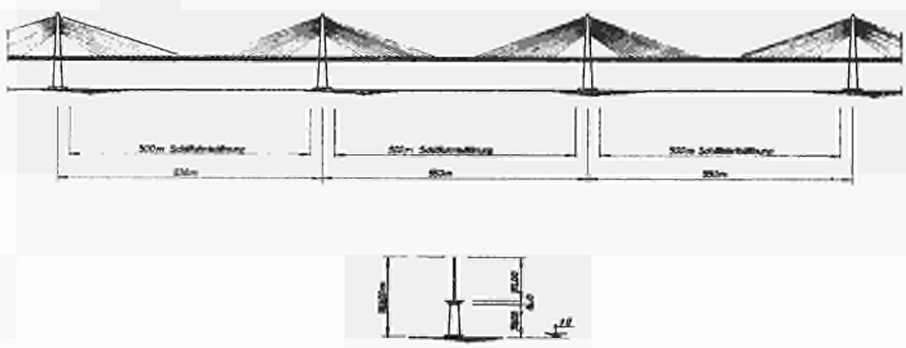

Fig. 16

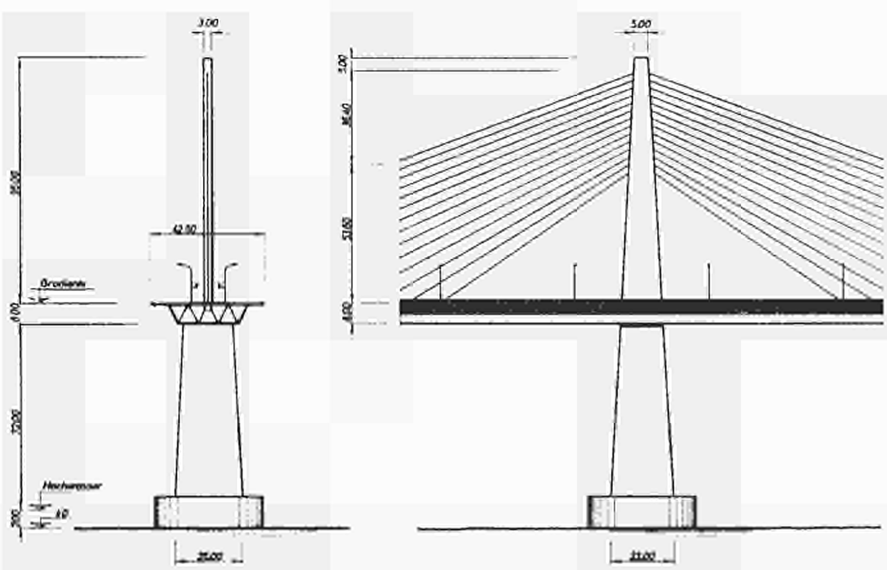

Fig. 17

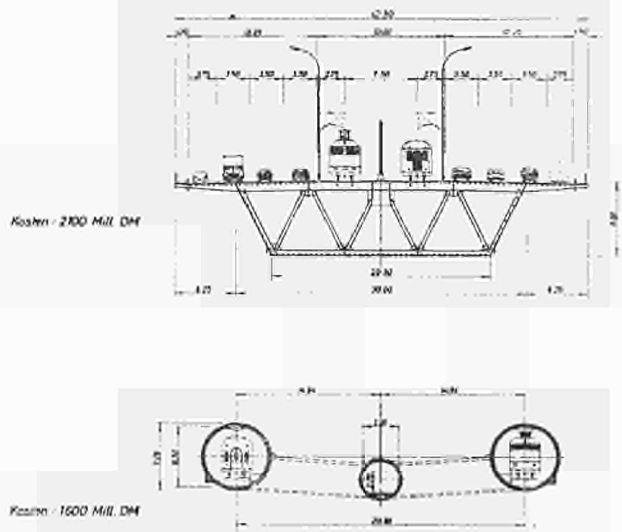

Fig. 18

Figure 17 shows the pylon of the bridge and the typical format of the ground based piers. All connections between the pylon, stiffening girder and cables will be rigid connections.

Figure 18 compares the cross-sections of the bridge and the tunnel proposal, and it is immediately obvious that the bridge can cope with a far greater volume of traffic than can the tunnel.

We have tried to evaluate the costs of my proposed bridge and to compare them with the costs estimated by the Anglo-French committee for the tunnel. It was found that the bridge, in spite of its manifold increase in carrying capacity, would only be 30% dearer than the proposed tunnel. Moreover it is well known that tunnelling jobs on account of the difficult foundation conditions have usually, in the last decade, transpired to be considerably dearer than the original estimate suggested. For these reasons it should still be possible to convince the interested governments of the suitability and profitability of a bridge over the Channel. This is a genuine problem for all authorities interested in steelwork and the application of steel. By working together at a European level it would be possible to erect for the permanent Channel crossing, a bridge which would take an economic and suitable form and which, at the same time, would be an imposing structure.

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L. ANSELMINI

A New Stainless Steel Bridge Beam Development

(Original language: English)

Introduction

The road and bridge building activity brought about by the intensive highway improvement programs in the United States and Europe has led to a search for new bridge designs incorporating lower construction and maintenance costs.

Stainless steel with its high strength and modulus of elasticity, excellent corrosion resistance, fabricability and appearance is a material of construction worthy of consideration for obtaining these goals.

Dr. Giorgio Baroni was commissioned by The International Nickel Co., Inc., to design a composite bridge using stainless steel beams. The economic requirements dictated a maximum of 7 pounds of stainless steel per square foot of roadway, and these limits were imposed on the design. The beams are designed as part of a composite steel-concrete bridge structure with a 64 foot span having a 9 foot spacing between beams and capable of operating under an American Association of State Highway Officials H 20-S16-44 highway loading. This is the heaviest American highway loading and consists of a 20 ton gross weight tractor truck with a 16 ton gross weight semi-trailer in each lane.

Baroni developed a new type beam, as revolutionary in concept as the diagonal-tension type beam shown in photo 1 and without the disadvantage of visible waves in the webs under working loads. The beam consists of a web of two concave Type 301 (17% Cr, 7% Ni) stainless steel sheets 0.070 inches thick mounted back to back with channel-like Type 301 stainless steel top and bottom flanges, as shown in photo 2.

The construction of the beam is illustrated in Figure 1 which shows a cross-section, including the corrosion resistant high-strength low-alloy steel (typical composition 0.09% C,

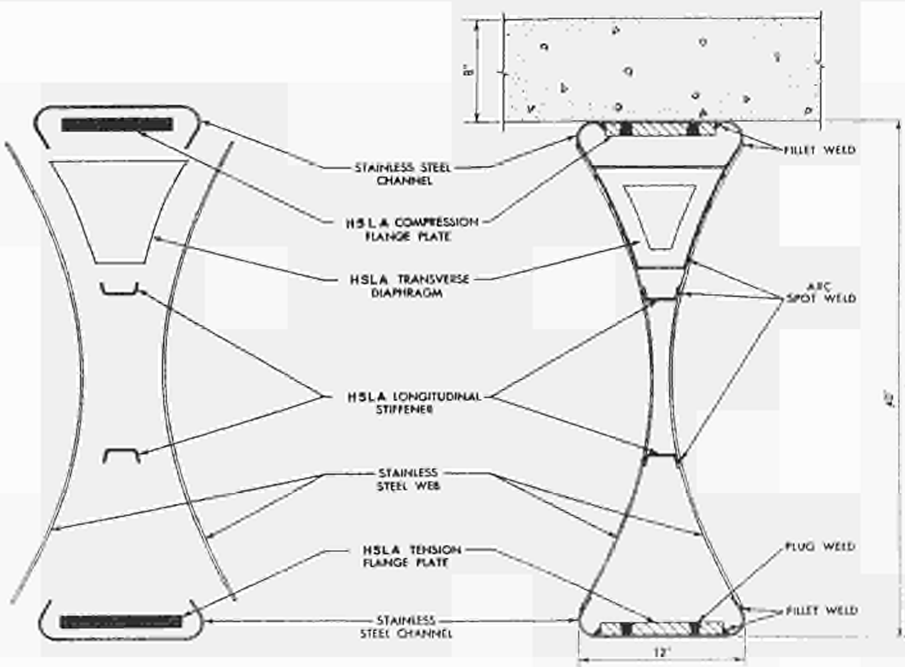


Fig. 1

0.41% Cu, 0.84% Cr, 0.28% Ni) flange plates used to bring the total flange area of stainless, plus high-strength low-alloy steel to the required size. Economic considerations limit the quantity of stainless steel used in the flanges.

The longitudinal stiffeners shown in the cross-section divide the web into smaller panels to increase the critical stress or buckling stress of the web. The number and position of the longitudinal stiffeners determine the depth of the panels of the web.

It is evident from the cross-section of the beam that it has excellent lateral stability and torsional rigidity since the compression flange is supported for its entire length and the enclosed area of the cross-section is large.

Full depth transverse ribs are required at the reaction points of the beam and at locations where a large single force is applied such as connection points of external diaphragms. Transverse diaphragms might also be useful if it is desirable to shorten the length of the web panel to increase the critical stress.

The determination of the allowable stresses in the tension and compression flanges was made by conventional methods. The stainless steel sheet used had a yield strength of 50,000 psi and therefore, is compatible with the flange of corrosion-resistant high-strength low-alloy steel which has a yield strength of at least 50,000 psi.

For calculation of the critical stress in the web, the equations and factors for combined bending and shear published in Timoshenko and Gere "Theory of Elastic Stability" [1] were used. The k-factor to be used is determined by the ratio of the shear stress to the critical shear stress as given in Table 9-10 of the same reference[2]. The calculated critical stress applies to a flat plate with supported edges. However, the edges of the actual web plate have some restraint, hence the critical stress will be conservative and the critical stress of a curved plate is higher. How much higher can be seen from equation (87) of NACA, Part III-"Handbook of Structural Stability"[3]

τ_{CR} curved plate $= 0.37\ \tau_{CR}$ flat plate $(Z_b)^{\frac{1}{2}}$.

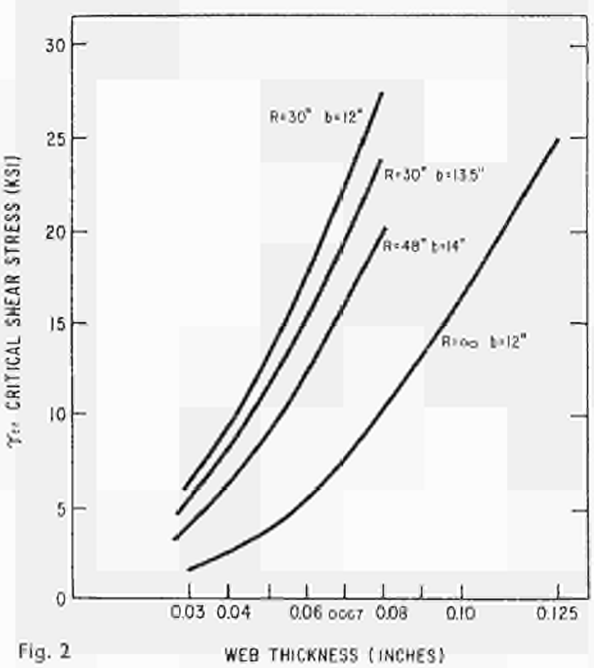


Fig. 2

Taking the lowest curvature factor $Z_b = 30$ for which this equation applies, the critical stress of a curved plate is at least twice that of a flat plate under the same panel dimensions and border conditions. The curves in figure 2 illustrate this point. For a constant panel depth (b = 12"), the lower curve shows the critical shear stress of flat webs ($R = \infty$) for different web thicknesses while the upper curves show the effect of web curvature of different radii.

In NACA, Part I—"Handbook of Structural Stability", [6] equations are also given for critical stress for combined shear and bending of flat panels. With the help of equation (37) NACA Part III—"Handbook of Structural Stability" [3], the critical stress of a curved panel is determined for shear and bending.

Combined shear and compression is encountered in the upper part of the longitudinally divided web. A series of curves for determining the critical stress in curved panels under combined shear and compression or tension taken from a publication of A. Kromm can be found in the aforementioned Timoshenko and Gere "Theory of Elastic Stability" [1].

Tests

Design calculations have been established for the stainless steel beam and the composite beam, i.e. a stainless steel girder with a concrete deck working together as in a bridge. The curved webs of the Baroni design constitute a new element in civil engineering, however, there are sufficient related theoretical and test data to calculate the stress level in the webs. To determine if deviations from the established calculations for the curved webs were needed, INCO decided to make and test a beam. It was also considered desirable to check the behavior of the fabrication under vibrating loads as well as to confirm the shear connector calculations with regard to their dimensions and placement along the beam.

A test beam 30 feet long, 40 inches deep and 12 inches wide was constructed by Pittsburgh-Des Moines Steel Company, Pittsburgh, Pennsylvania. The test beam was designed to simulate stress conditions imposed by an American Association of State Highway Officials H20-S16 highway loading on a 64 foot long highway bridge.

The Fritz Engineering Laboratory at Lehigh University, Bethlehem, Pennsylvania, tested the beam in a series of static tests on the beam alone and in static and fatigue tests on the beam composite with a concrete deck. Static tests of the steel beam in the normal (see photo 3) and inverted position revealed that buckling of the web plates did not occur at full working loads. There was virtually no difference in the action of the beam in an inverted position, which suggests that the transverse diaphragms shown in figure 1 could be omitted. The dashed curve of Figure 3 shows the behavior of the beam while inverted and with no transverse diaphragm on the compression side of the beam above the neutral axis.

Strain measurements indicated that all longitudinal parts of the cross-section of the member were effective and that the composite beam shown in photo 4 acted predictably with respect to slip, strains and deflections.

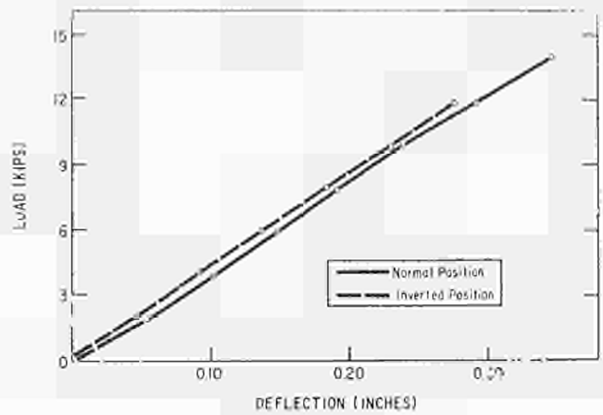


Fig. 3

Fatigue testing consisting of 600,000 cycles with the stress range varying from 10% to 100% of full load stress, was carried out successfully. Arc-spot welded specimens of the same alloy and gage combinations used in the beam were fatigue tested at Ecole Polytechnique, Montreal, Canada. Minimum curves such as that shown in figure 4 indicate maximum cyclic pull-pull loads of 750 pounds for 3 million cycles for stainless steel to high-strength low-alloy steel arc-spot weld connections.

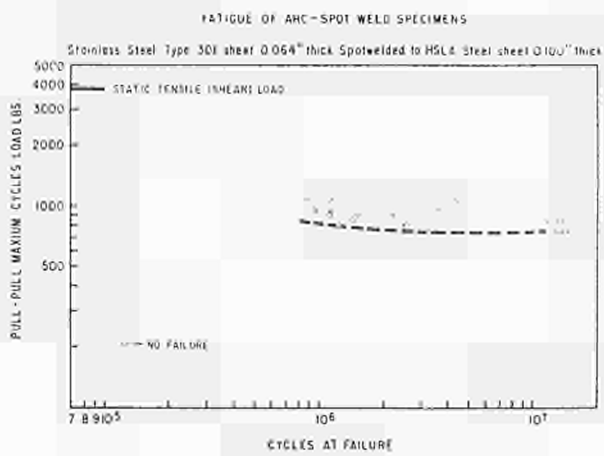


Fig. 4

Finally, the composite member was tested to destruction in Lehigh's 5,000,000 pound Universal Testing Machine, as shown in photo 5. A load corresponding to 6.2 times the full working load was applied before failure occurred. The load-deflection curve from this test, shown in figure 5, exhibits straight-line, elastic action up to and well above the working range.

Stress calculations

A review of the calculations and a comparison of the analytical values with the test results follows and is based on measurements made at Section I, and Section II, shown in figure 6, during the ultimate load test of the composite beam with :

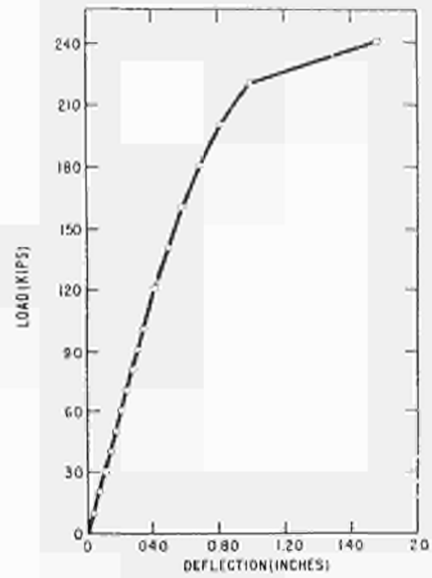


Fig. 5

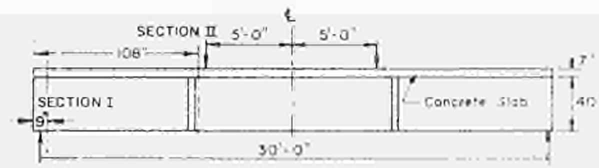


Fig. 6

Dead Load = W = 22,980 pounds
Live Load = P = 240,000 pounds.

The following constants were used in calculations of the critical or buckling stress of the web :

E	= Modulus of Elasticity	$= 29 \times 10^6$ psi
m	= Poisson's Ratio	= 0.33
b	= Depth of panel	= 12"
R	= Curvature radius	= 34"
t	= Thickness of web	= 0.09" at the ends and 0.07" in the middle part of the beam
h	= Depth of web	$= 3 \times b = 3 \times 12'' = 36''$

The length of the web panel is much longer than the width resulting in lower k-factors and therefore in lower critical stress. The 36 inch deep webs are divided by longitudinal stiffeners into three panels of 12 inches each.

In the ultimate test of the composite beam the shear stresses in the web were:

Section I, 9" from beam end. $t_I = 0.09''$;

τ_I = Shear stress in web at Section I

$$\tau_I = \frac{P/2 + V_1}{2 \times H \times t_I}$$

where

V_I = Shear force at Section I = 10930 pounds

$$\tau_I = \frac{120000 + 10930}{2 \times 36 \times 0.09} = \mathbf{20100\ psi}$$

Section II, 108″ from beam end (near joint between 0.09″ and 0.07″ web), $t_{II} = 0.07''$;

τ_{II} = Shear stress in web at Section II

$$\tau_{II} = \frac{P/2 + V_{II}}{2 \times H \times t_{II}}$$

where

V_{II} = shear force at Section II = 4635 pounds

$$\tau_{II} = \frac{120000 + 4635}{2 \times 36 \times 0.07} = \mathbf{24700\ psi}$$

But the upper web panel of the stainless steel beam is also in tension due to bending. The tension in Section I increases from 200 psi at the top of the panel to 1500 psi at the bottom of the panel and in Section II from 2100 to 14730 psi. The tension of the panel increases the critical stress of the panel. For the computation of the critical stress of the panel the tension stress is taken to be the arithmetic mean of the top and bottom stresses, *i.e.*

For Section I $\quad \sigma_{XI} =$ **850 psi**
For Section II $\quad \sigma_{XII} =$ **8420 psi**

Calculation of the critical stress in the web was checked by several methods, namely:

A. Shear stress only according to formulas from NACA, Part II. — "Handbook of Structural Stability" [5].

B. Shear and tension according to reference of A. Kromm in Timoshenko and Gere "Theory of Elastic Stability" [3].

C. Shear and tension according to NACA, Part II. — "Handbook of Structural Stability" [5].

A. Using equations for shear stress only published in NACA, Part II. — „Handbook of Structural Stability" [5]:

Formula (85) $\tau_{CR} = K_S \dfrac{\pi^2 E}{12(1 - m^2)} \left(\dfrac{t}{b}\right)^2$

designating the constant

$$D = \frac{\pi^2 E}{12(1 - m^2)} = \frac{\pi^2 \times 29 \times 10^6}{12(1 - 0.33^2)} =$$

26.8×10^6 psi

$$B = \left(\frac{t}{b}\right)^2$$

τ'_{CR} = critical stress for simply supported edges
τ''_{CR} = critical stress for clamped edges
K'_S = factor for simply supported edges
K''_S = factor for clamped edges

K_S is determined by the curvature parameter

$$Z_b = \frac{b^2}{R \times t}(1 - m^2)^{\frac{1}{2}}$$

For Section I

$$Z_{bI} = \frac{12^2}{34 \times 0.09}(1 - 0.33^2)^{\frac{1}{2}} = 42$$

and $B_I = \left(\dfrac{t_I}{b}\right)^2 = \left(\dfrac{0.09}{12}\right)^2 = 56.2 \times 10^{-6}$

for simply supported edges $\dfrac{a}{b} = \infty$ (long panel) and

$Z_{bI} = 42$, $K'_S = 12$ according to figure 49a

$\therefore\ \tau'_{CR} = K'_S \times D \times B = 12 \times 26.8 \times 10^6 \times 56.2 \times 10^{-6} =$ **18100 psi**

for clamped edges figure 49b shows $K''_S = 23$

$\therefore\ \tau''_{CR} = K''_S \times D \times B_I = 23 \times 26.8 \times 10^6 \times 56.2 \times 10^{-6}$ = **34700 psi**

For Section II $Z_{bII} = \dfrac{12^2(1 - 0.33^2)^{\frac{1}{2}}}{34 \times 0.07} = 54$

and $B_{II} = \left(\dfrac{t_{II}}{b}\right)^2 = \left(\dfrac{0.07}{12}\right)^2 = 34.1 \times 10^{-6}$

for simply supported edges $K'_S = 13.5$ and
$\therefore\ \tau'_{CR} = K'_S \times D \times B_{II} = 13.5 \times 26.8 \times 10^6 \times 34.1 \times 10^{-6}$ = **12350 psi**

for clamped edges $K''_S = 25$ and

$\therefore\ \tau''_{CR} = K''_S \times D \times B_{II} = 25 \times 26.8 \times 10^6 \times 34.1 \times 10^{-6}$ = **22850 psi**

B. Considering additionally the tension in the panel, Kromm's curves for simply supported edges are used. The curves are shown by Timoshenko and Gere "Theory of Elastic Stability", [3]. The abscissa of these curves is $\sqrt[4]{\omega}$ where :

$$\sqrt[4]{\omega} = \frac{b}{\pi \times R}\left(\frac{R}{t}\right)^{\frac{1}{2}}\left[12(1 - m^2)\right]^{\frac{1}{4}}$$

The ordinate of these curves is the ratio of τ_{CR} to σ_{CR} the latter value being calculated for long compressed rectangular plates as follows :

$$\sigma_{CR} = \frac{\pi^2 E}{3(1 - m^2)}\left(\frac{t}{b}\right)^2 = 4 \times D \times B$$

and σ_X is the tension or compression stress in the panel.

For Section I the abscissa is :

$$\sqrt[4]{\omega} = \frac{12}{\pi \times 34}\left(\frac{34}{0.09}\right)^{\frac{1}{2}}\left[12(1 - 0.33^2)\right]^{\frac{1}{4}} = 3.96$$

and $\sigma_{CR} = 4 \times 26.8 \times 10^6 \times 56.2 \times 10^{-6} =$ **6020 psi**

and $\dfrac{\sigma_{XI}}{\sigma_{CR}} = \dfrac{850}{6020} = 0.14$

Corresponding to these figures the curves show

$\dfrac{\tau_{CR}}{\sigma_{CR}} = 3.3$ and $\tau'_{CR} = 3.3 \times 6020 =$ **19870 psi**

To determine the critical stress for clamped edges the relation between the k-factors for clamped and simply supported edges is used, the k-factors taken from Timoshenko and Gere "Theory of Elastic Stability", [4] equation (9-9) and equation (9-10) — for long flat plates.

$$F = \frac{8.98}{5.35} = 1.68$$

$\therefore \ \tau''_{CR} = 1.68 \times 19870 = \mathbf{33300\ psi}$

For Section II

$$\sqrt[4]{\omega} = \frac{12}{\pi \times 34}\left(\frac{34}{0.07}\right)^{\frac{1}{2}}\left[12\,(1 - 0.33^2)\right]^{\frac{1}{4}} = 4.5$$

$\sigma_{CR} = 4 \times 26.8 \times 10^6 \times 34.1 \times 10^{-6} = \mathbf{3650\ psi}$

$$\frac{\sigma_X}{\sigma_{CR}} = \frac{\sigma_{XII}}{\sigma_{CR}} = \frac{8420}{3650} = 2.32$$

the curves show $\dfrac{\tau}{\sigma_{CR}} = 4.0$

$\therefore \ \tau'_{CR} = 4 \times 3650 = \mathbf{14600\ psi}$

and $\tau''_{CR} = 1.68 \times 14600 = \mathbf{24500\ psi}$

C. According to NACA, Part III, „Handbook of Structural Stability", [5] the interaction equation (89) for shear and tension is $R^2_S - R_X = 1$; R_X is negative according to the designations of the "Handbook". [5]

Rearranged $R_S = (1 + R_X)^{\frac{1}{2}}$

$$\text{where } R_S = \frac{\tau_{CR}\ \text{combined}}{\tau_{CR}\ \text{shear}}$$

$$R_X = \frac{\sigma\ \text{tension}}{\sigma_{CR}\ \text{compression}}$$

σ_{CR} compression $= K_C \times D \times B$

τ_{CR} shear $= K_S \times D \times B$

For Section I

$K_C = 30$ according to Figure 38a of NACA, Part III, „Handbook of Structural Stability" [5]

and σ_{CR} compression $= 30 \times 26.8 \times 10^6 \times 56.2 \times 10^{-6} =$ **45200 psi**

$$R_X = \frac{\sigma_{XI}}{\sigma_{CR}\ \text{compression}} = \frac{850}{45200} = 0.02$$

and $R_S = (1 + 0.02)^{\frac{1}{2}} = 1.01$

according to figure 49a of NACA, Part III, „Handbook of Structural Stability" [5]

$K_S = 12$

$\therefore \ \tau_{CR}$ shear $= 12 \times 26.8 \times 10^6 \times 56.2 \times 10^{-6} =$ **18100 psi**

$\tau'_{CR} = 18100 \times 1.01 = \mathbf{18300\ psi}$

or clamped edges $K_c = 23$

$\therefore \ \tau_{CR}$ shear $= 23 \times 26.8 \times 10^6 \times 56.2 \times 10^{-6} =$ **34700 ps**

$\tau''_{CR} = 34700 \times 1.01 = \mathbf{35000\ psi}$

For Section II

Simply supported edges $K_c = 40$

σ_{CR} compression $= 40 \times 26.8 \times 10^6 \times 56.2 \times 10^{-6} =$ **36700 psi**

$$R_X = \frac{\sigma\,XII}{\sigma_{CR}\ \text{compression}} = \frac{8420}{36700} = 0.229$$

and $R_S = (1 + 0.229)^{\frac{1}{2}} = 1.135 \quad K_S = 13.5$

τ_{CR} shear $= 13.5 \times 26.8 \times 10^6 \times 34.1 \times 10^{-6} =$ **12300 psi**

$\tau'_{CR} = 12300 \times 1.135 = \mathbf{14000\ psi}$

Edges clamped $K_S = 25$

τ_{CR} shear $= 25 \times 26.8 \times 10^6 \times 34.1 \times 10^{-6} =$ **22800 psi**

$\tau''_{CR} = 22800 \times 1.135 = \mathbf{25800\ psi}$

The following table shows the comparison of actual and calculated critical stresses in psi using the methods outlined:

Method	Section I		Section II	
	Panel edges			
	Simply Supported	Clamped	Simply Supported	Clamped
Measured Test Stresses, psi	20100		24700	
a) Shear only "Handbook", Part III, psi	18100	34700	12350	22850
b) Shear + Tension Kromm's curves, psi	19870	33300	14600	24500
c) Shear + Tension "Handbook", Part III, psi	18300	35000	14000	25850

The ultimate test showed some slight elastic buckling of the web at the top of the Section II just below the concrete slab. This elastic buckling does not mean failure of the beam, since it will disappear as soon as the overload is removed. The amount of additional stress the webs will take in the plastic range can also be calculated according to NACA, Part III, „Handbook of Structural Stability"[5]. In the design, the allowable stress is reduced in relation to the critical stress by the safety factor. Our test showed that the available calculation methods are on the safe side.

After the concrete slab was removed, inspection of the shear connectors showed no distortion or breaking, assuring that they were adequately dimensioned and spaced.

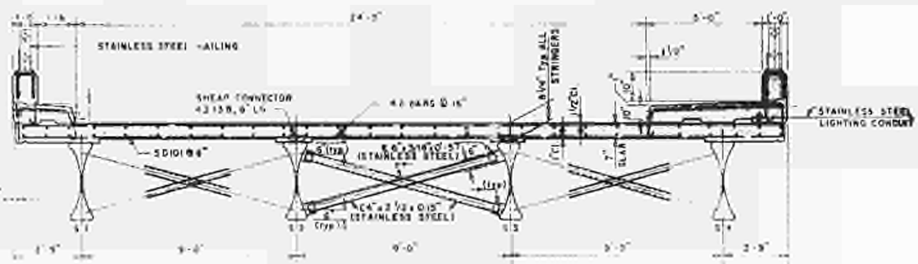
Cost comparisons and prototype development

Costs have been analyzed for building a highway bridge of cross-section shown in figure 7 using stainless steel beams. The light gage stainless steel webs and flanges keep the material costs comparable to similar bridges constructed of other materials. The design requires only 4.6 pounds of stainless steel and 6.9 pounds of high-strength low-alloy steel per square foot of roadway.

Manufacturers skilled in the efficient fabrication of light gage stainless steel structures estimate fabrication costs closely approaching those of competitive materials. Refinements in assembly details are expected to reduce fabrication costs still further. Freight, erection, roadway and side walk costs of a bridge using stainless steel girders were found in a recent study to be comparable to those of other girder materials.

Bridges built using stainless beams would never require painting and would have a most attractive appearence as shown in photograph 6.

In summary, tests have confirmed the technical adequacy of the design and our economic studies show promise of a favorable competitive position. Therefore, the time is appropriate for consideration of prototype, maintenance-free stainless steel highway over-pass bridges with esthetically appealing clean lines. Toward this end, discussions with various State and local authorities are now going forward, and we expect that arrangements for the construction of an experimental bridge will be concluded in the near future.



TYPICAL CROSS SECTION

Fig. 7

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[1] Timoshenko and Gere, "Theory of Elastic Stability", second edition, pp. 384-385, McGraw-Hill Book Company, Inc., New York, N.Y., 1961.

[2] Timoshenko and Gere, ibid, p. 382.

[3] Timoshenko and Gere, ibid. pp. 489-490.

[4] Timoshenko and Gere, ibid. pp. 483-484.

[5] Gerard and Becker, "National Advisory Committee for Aeronautics Technical Note 3783, Handbook of Structural Stability, Part III-Buckling of Curved Plates and Shells", 1957.

[6] Gerard and Becker ,,National Advisory Committee for Aeronautics Technical Note 3781, Handbook of Structural Stability, Part I—Buckling of Flat Plates", 1957.

Description of photographs

1 — Diagonal-tension type beam.

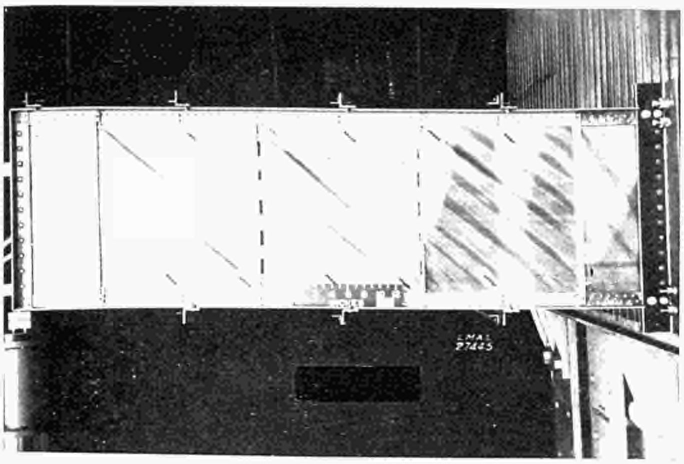
2 — New type beam developed by Baroni.

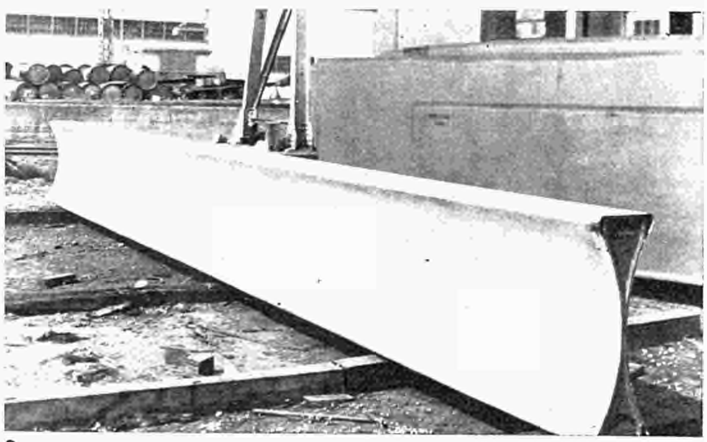
3 — Static tests of the steel beam.

4 — Testing of composite beam.

5 — Destruction test of the composite beam.

6 — Stainless steel bridge.





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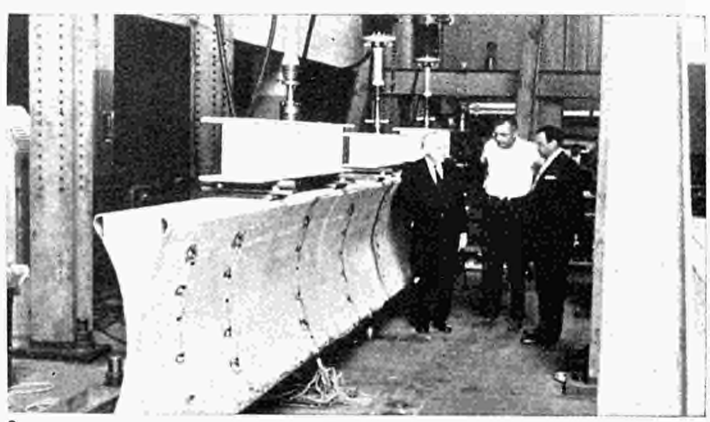

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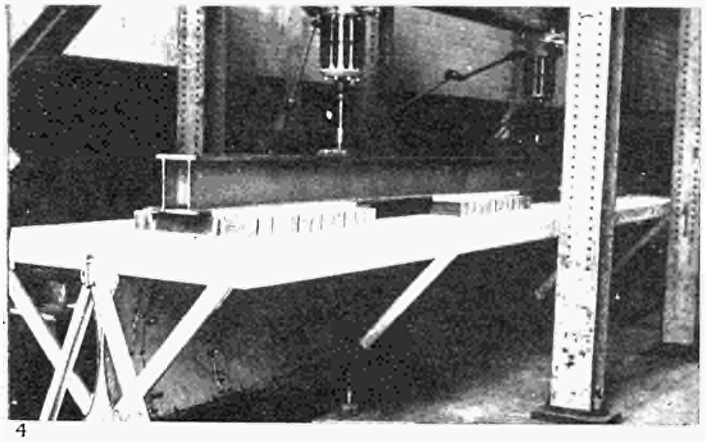

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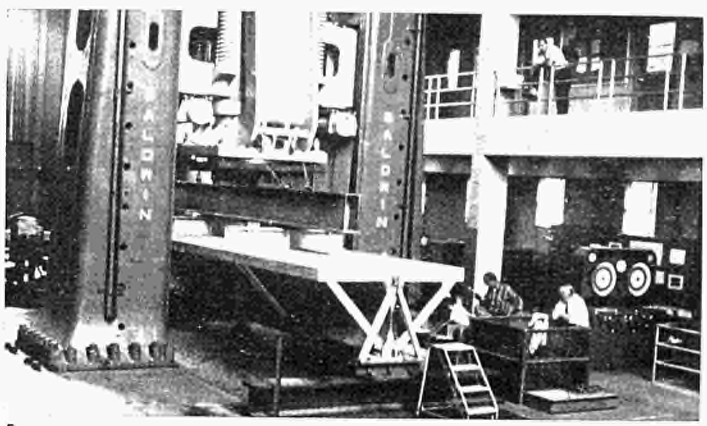

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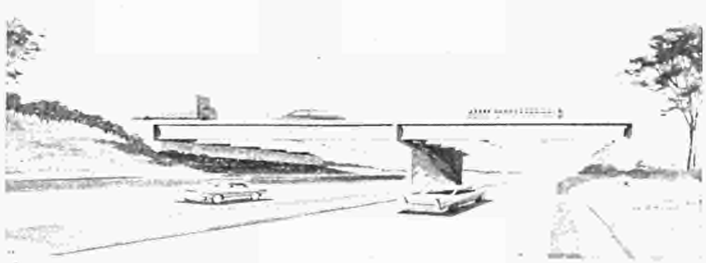


6

Gavin Burton STEWART

Tubular Structures in Bridges

(Original text: English)

The discussions of this working party on bridges would be very incomplete if the part played by the steel tube in bridge construction were not mentioned and I feel called upon, as President of the international body which champions the structural tube, to recall very briefly the part played by the tube in bridge construction and to point to its present great potential in that field.

The early railway bridges which can still be found in Britain, and of which we are extremely — if somewhat nostalgically — proud, were in many cases either built as square section tubes or were suspended from large diameter round tubes working in compression.

Shown in photograph 1, without any apology whatsoever, is the Saltash Bridge associated with the great name of Brunel, which still carries rail traffic over the Tamar River.

In photograph 2, the most famous of all bridges, the Forth Bridge, now seventy-five years old and the supreme engineering expression of its age.

The main structural members of these justly-renowned bridges are tubes. And for good reasons. The tube is the ideal compression and torsion element. A tubular structure is invariably lighter than its equivalent in rolled sections — something which matters very much in bridges. And so does the reduction in wind load obtained from the circular shape. Maintenance is also easier, since the tube offers a smooth, even surface to treat against corrosion and the enclosed internal surface is ideally protected.

Why, with all these advantages, are there still so few tubular bridges? The answer is twofold. First, there is the deterrent of a slightly more expensive basic section, which tends to conceal the overall advantages of this form of construction, and, second, there is the backwash of an age when only large diameter tubes could be effectively joined by riveting. With modern welding technique the picture is, of course, reversed, and the welded tubular joint is superlative in its simplicity, neatness and engineering qualities (3).

This slowly growing awareness of the qualities of the tube as a structural element in bridge construction has led to some striking recent applications, some of which I propose to show.

First of all is this splendid road bridge over the Askwröfjorden in Sweden, which was opened in 1960 (4). This belongs still to the large diameter arched tube tradition, but the proportions are vast, the span is of the order of 300 metres and the tubes are almost 4 metres in diameter.

The great bridge at Hamburg is suspended from the tubular arch — a remarkably clean and handsome construction (5).

Also arched, but of lattice construction, is the service bridge at Duisburg, which is used for carrying mains over a waterway (6).

Most spectacular among service bridges is this no doubt familiar series of suspension pipe bridges with its quadruple catenaries built to carry an oil main over the Pô near Cremona in Italy. Photographs 7, 8 and 9 show the slender piers and the great cantilevers. The lightness and low wind drag of tubular construction scores most forcibly.

A most interesting development in bridge design is the recent evolution of the "Nielsen" bridge with twin arches leaning towards each other and joining at the top.

In photograph 10 an English example in tubular construction spanning a canal. This is an all welded construction.

Photographs 11 and 12 show a similar example, this time over the Rhône in Switzerland. Similar bridges, built in Italy, are made of large prefabricated sections which are factory welded and then bolted on site. Flanged tubes lend themselves admirably to this increasingly popular site-bolted form of construction.

Structural tubes have found another very important application in conveyor bridges and gantries. Photograph 13 illustrates one of a large number of examples in England.

Probably the most important use of tubes in bridges comes in fact from the one kilometer long chain of conveyor bridges at the Italsider Steel producing centre at Taranto, Italy. This drawing shows the uniform lattice construction of these bridges, reinforced for the longer spans by a Langer type arch (Fig. 1 and 2).

A completely different tubular application which should interest us, since we are particularly concerned with road bridges, is the use of concrete-filled tubes as pillars as shown in photographs 14 and 15 of a motorway intersection in England. The international organisation to which I shall be referring in a moment is carrying out research into these concrete-filled tubes and these are showing the greatest promise for this application.

Let us remember, finally, that the tube adds simplicity and elegance to its sterling engineering qualities. This aspect is happily gaining in importance, and the tube may well be of the greatest service in this respect to steel construction when it comes into competition with the flowing lines of reinforced concrete.

Photographs 16, 17 and 18 show a few pedestrian bridges built in tube.

And to show that the tube can have modest as well as spectacular applications in bridge structures, I refer to this intimate little picture of a footbridge in a Hanover park (19).

It is particularly appropriate that, in conclusion, I should mention in this great international setting that the leading tube makers and tubular steel constructors of several countries have grouped themselves into an international organisation bearing the name of the Comité International pour l'Etude et le Développement de la Construction Tubulaire, abbreviated to C.I.D.E.C.T. This Committee aims at the thorough study of this form of construction and its presentation to the whole world.

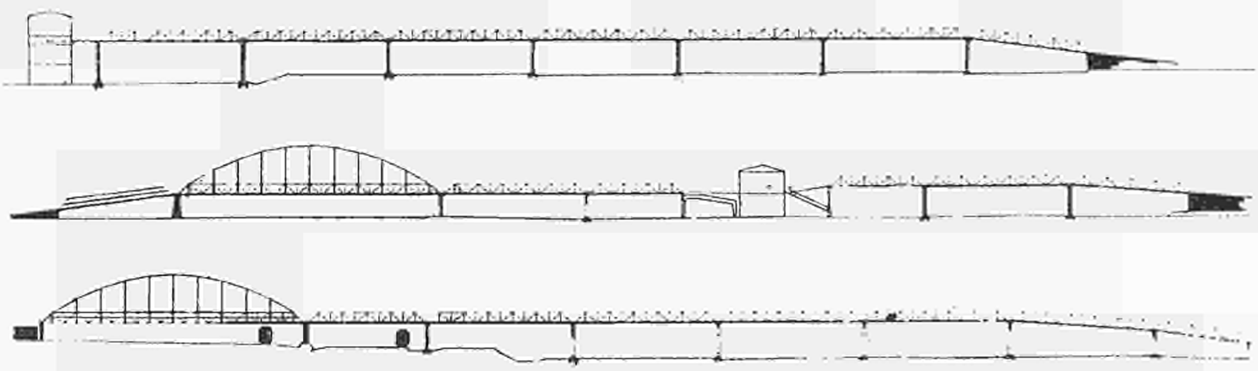


Fig. 1

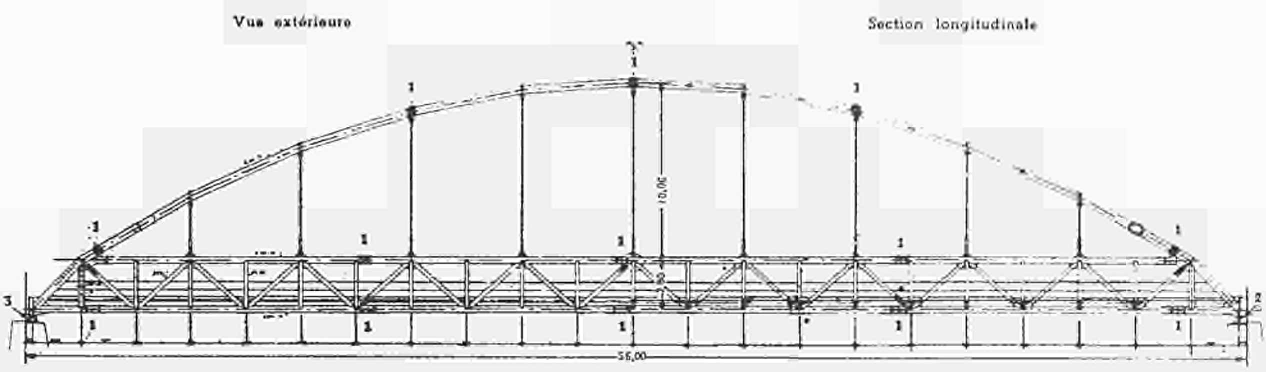
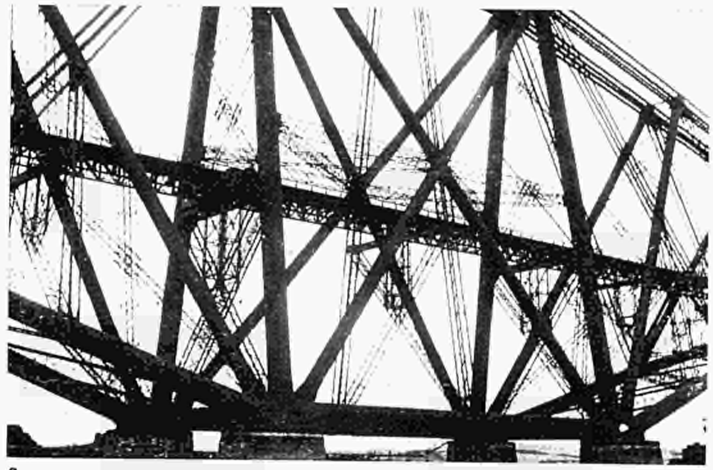


Fig. 2

Description of photographs

1 — Saltash Bridge.
2 — Forth Bridge.
3 — Welded tubular joint.
4 — Roadbridge over the Askwröfjord.
5 — Bridge at Hamburg.
6 — Service bridge at Duisburg.
7 - 9 — Suspension pipeline bridge over the Pô river near Cremona.
10 — An English example of tubular construction.
11 - 12 — Bridge over the Rhône river in Switzerland.
13 — Conveyor bridge in England.
14 - 15 — Model of motorway intersection in England.
16 - 18 — Pedestrian bridges.
19 — Footbridge in a park at Hanover.



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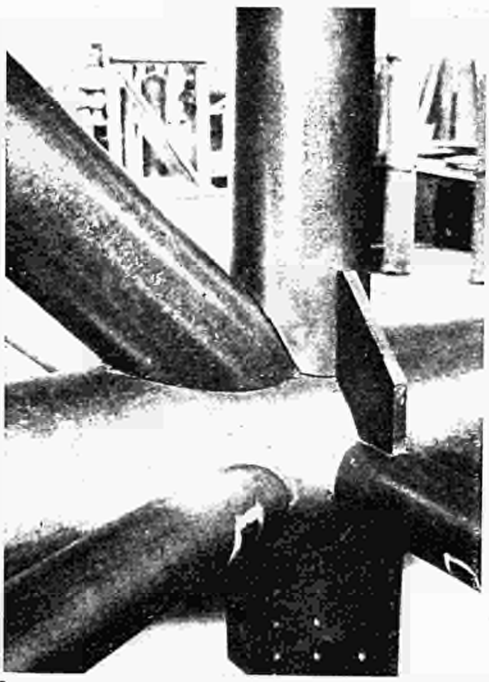

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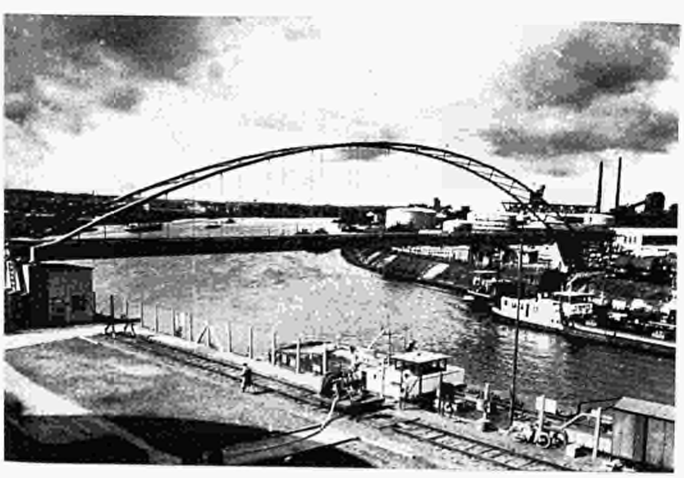

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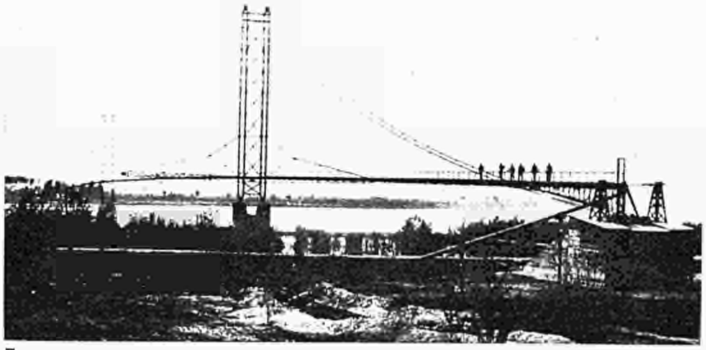
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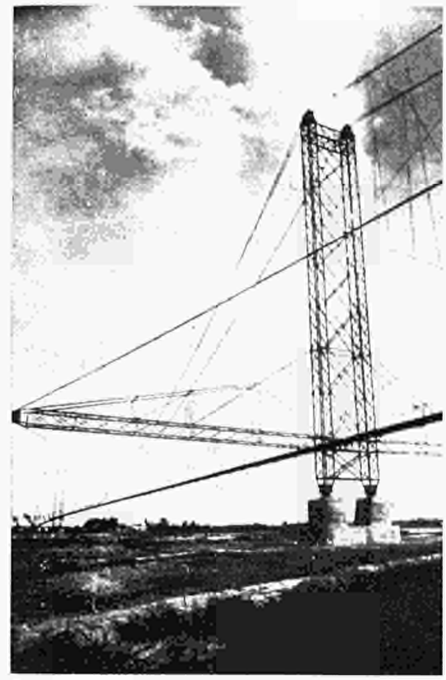
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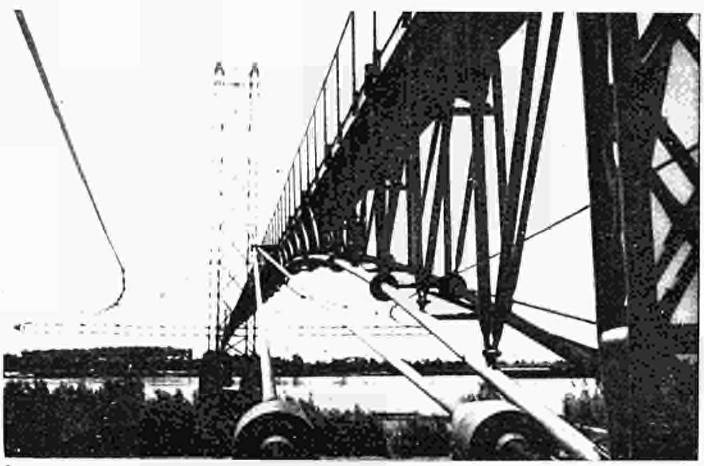
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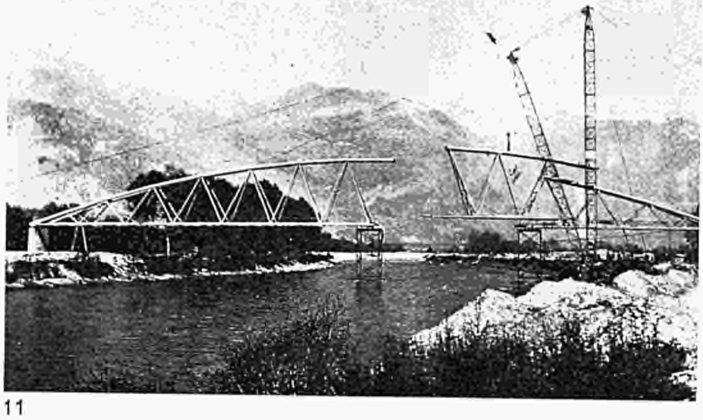
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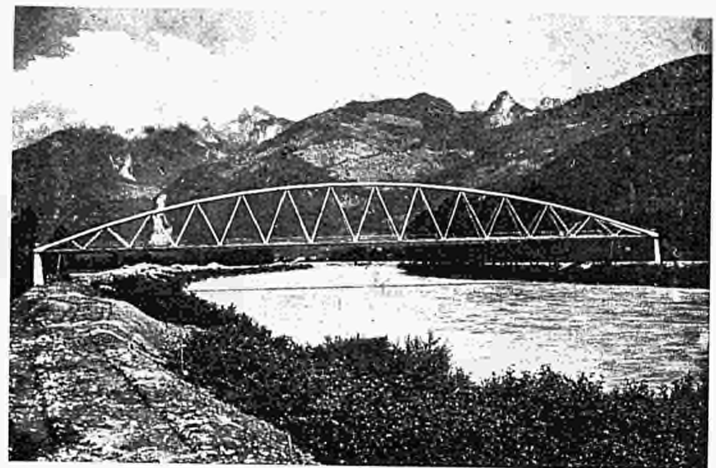
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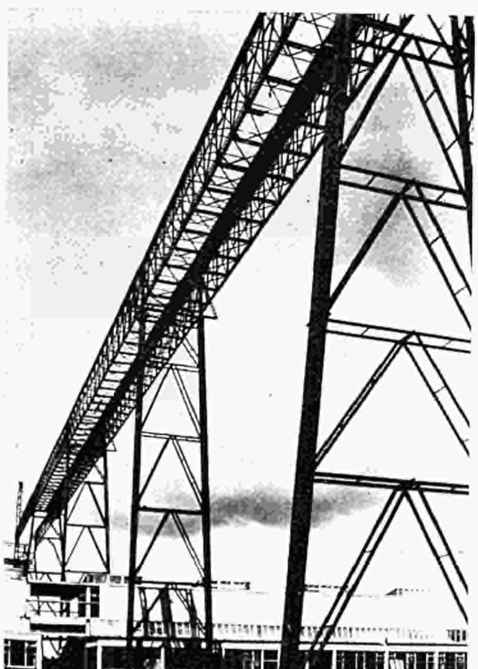
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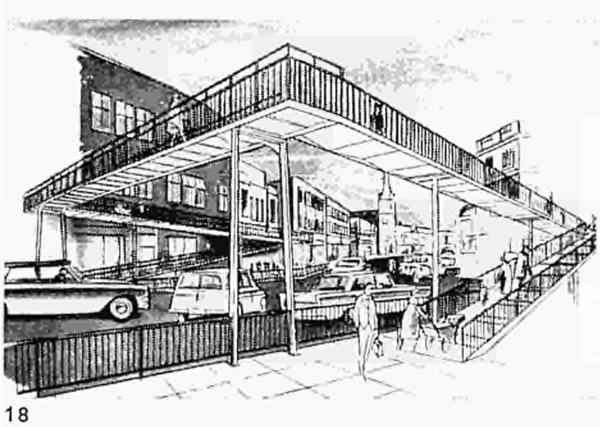

18

19

Pierre DUBAS

"La Madeleine" Bridge over the Sarine

(Original text: French)

A little way downstream from Fribourg the future Berne-Vevey motorroad crosses the Sarine Valley, which at this point is about 300 metres wide and about 50 metres deep. After the piers were built, the filling of the Schiffenen Dam created an artificial lake, 215 metres wide, at the site of the bridge.

The scheme finally adopted from among those submitted by a number of engineers, provides for a deck on continuous straigth girders of three spans of 85.50 m, 106.50 m and 85.50 m, two twin piers of hollow rectangular sections, which are very slender, as they are only 1.60 m in width, and two comparatively large abutments, slightly overhanging (1). This solution was not only the most economical but also had the advantage of involving a smaller number of piers — an important factor owing to the short time available before the Schiffenen hydro-electric station was to be put into service.

The structure comprises two separate decks, one for each direction of traffic. Under each deck, the two plate girders, suitably wind-braced and strutted (2), act compositely with the precast concrete deck through the medium of shear connectors (3). The deck comprises a central carriageway 7.75 m wide and two parking-strips each 1.25 m wide which together with the safety margins result in a total width of 11.05 m.

Although not of exceptional dimensions, this structure has certain special features to which attention should be drawn.

As regards the grades of steel used, it will be seen that the girders are mainly of St. 52 but that the four sections at the piers are of steel with a high yield point, H.O.A.G. 55 F.K.V.-5, of which the chemical composition (determined at the time of inspection and acceptance) is shown in the table. This is a high-alloy steel, delivered in the normalized condition, its guaranteed yield point being 40 kg/mm²; this was actually exceeded at the final approval test, even for the thickest plates (50 mm). Impact tests were also carried out on the material in the natural state (Charpy-V longitudinally, at −10°C, giving results above 10 kg/cm²), and after ageing (D.V.M. longitudinally, at 0°C, giving results above 7 kg/cm²) together with the Kommerell bending test.

Properties of steel HOAG 55

BRIDGE OVER THE SARINE

Steel with high yield point HOAG 55 FKV

CHEMICAL ANALYSIS

C	Si	Mn	P	S	Ni	Va
0.17	0.50	1.50	0.025	0.025	0.50	0.10 %

MECHANICAL PROPERTIES

Guaranteed yield point	40 kg/mm²
Actual yield point (at time of acceptance)	> 42 kg/mm²
Impact tests: Charpy-V, (at time of acceptance)	− 10°
(longitudinal) DVM, after ageing,	± 0°

Photograph 4 shows a section of the pier at the works with the top flange 850 mm by 45 mm, the bottom flange 850 mm by 50 mm and the web 14 mm thick, increased to 20 mm over the bearing. In the span, the web is only 10 mm thick, although it is more than 4 m in height; it is stiffened by four rows of box-section stiffeners of high flexural and torsional strength.

As for the cost of material, the use of steel with a high elastic limit has not resulted in any appreciable saving, primarily because the thickness of the web is basically determined by buckling considerations. On the other hand, the scheme gives a considerable reduction in the cross section of the flange plates, particularly in their thickness, and this is a favourable factor against brittle fracture. The bottom flange at the pier also has the same dimensions (*i.e* 850 mm by 50 mm) as the thickest plate in the span.

This reduction in the cross sections at the piers also reduces the tensile stresses caused in the concrete deck by shrinkage, and results in lower creep losses.

The problem of the tensile stresses in the concrete is of prime importance in this structure, which utilizes a deck consisting of precast units with plan dimensions of 11.05 × 2.00 m and weighing 13 t (5). The units are placed in position by a heavy fork truck running on planks placed diagonally. The joints between the elements, with their junction bars, will be concreted in such a way that the floor will become monolithic and to reduce the risk of cracking in the joints, it is intended to apply longitudinal pre-compression. For this purpose, the girders are first of all cambered by 160 cm at the piers (6). Once the joints in the region of the piers have been completed, the corresponding portion of the deck will be pre-stressed (1800 t) by cables inserted in the longitudinal sheaths. The remaining joints between units will then be made, and the connection between the steel and the concrete will be effected by concreting the recesses provided at intervals of one metre, opposite the shear connectors. This will be followed by the over-all pre-compression, by jacking down the girders by 160 cm at the piers, which corresponds to an initial moment of 8000 tm per bridge.

The girders have been transported in pieces about 20-24 m long (7), assembled on the bank by welding, sand-blasted, painted and then launched in successive stages. At the time of crossing the central bay, the rise of the up-stream cutwater was more than 3.50 m (8). To prevent torsional oscillations under the effect of the wind, the bridge included not only the final lower wind-bracing but also an upper erection wind-bracing system.

The total weight of the metal frame amounts to 610 t, including sill girders; that is, a little less than 200 kg/m².

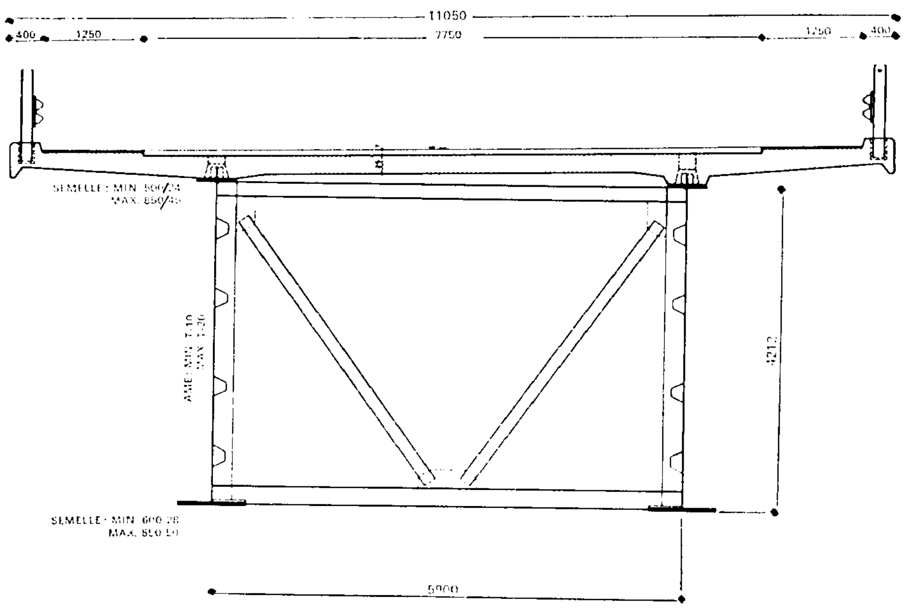

Fig. 1 Section of pier

Description of photographs

1 — General view of the bridge.
2 — Interior of girders.
3 — Underside of girders during erection.
4 — Transverse section through deck.
5 — Precast units before placing.
6 — Steelwork after launching.
7 — A girder section leaving the workshops.
8 — Launching the first and second piers.

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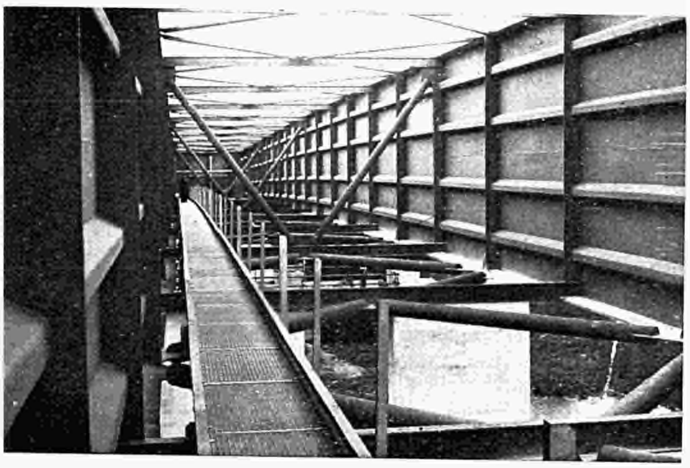

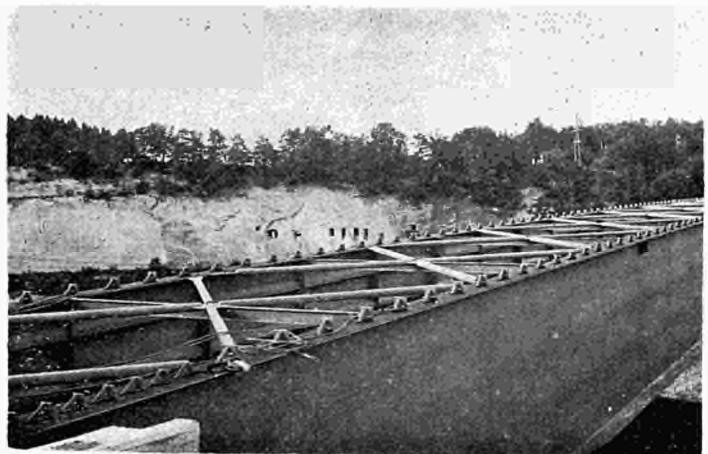

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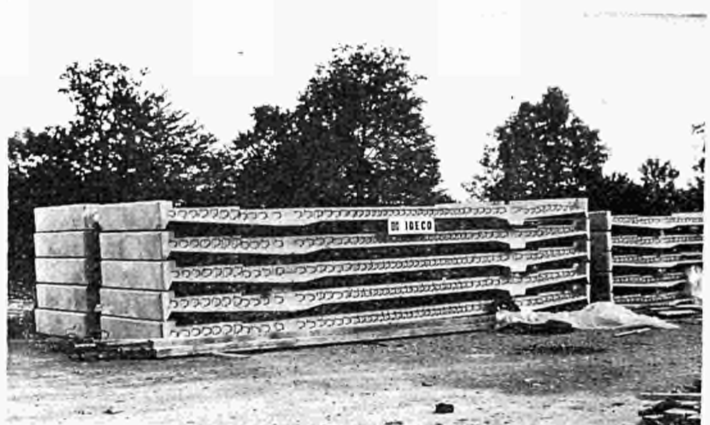

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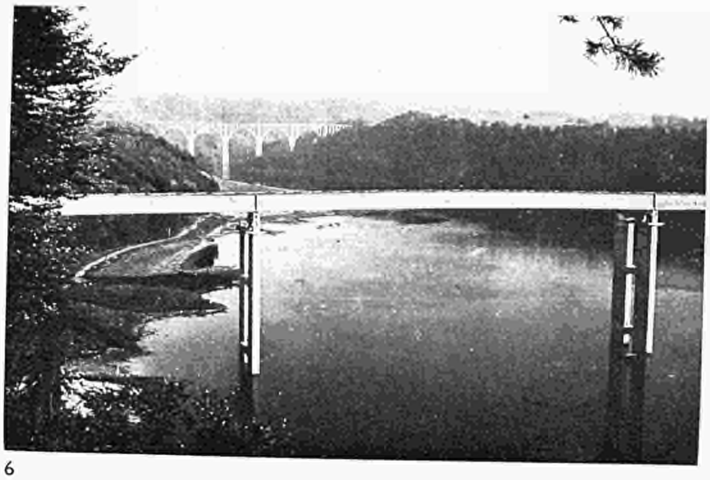
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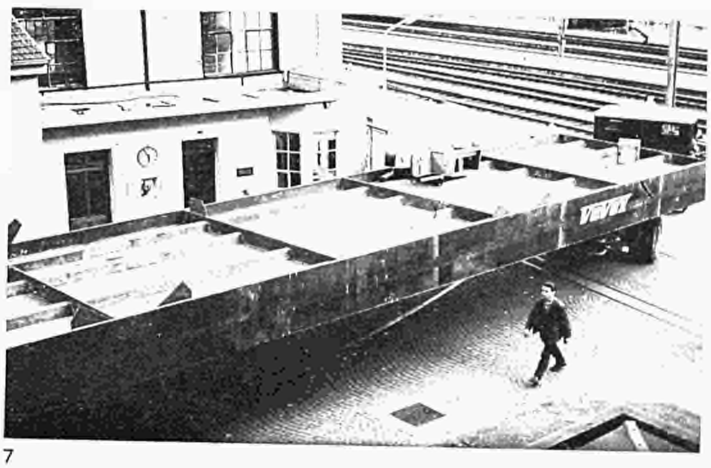
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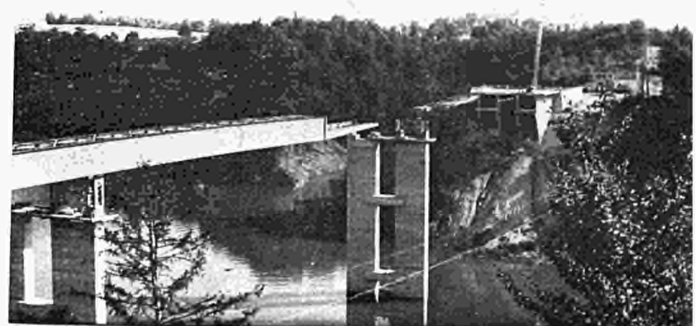
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Azarius DOBRUSZKES

The Contribution made by the Preflex System to the Development of Steel Bridges

(Original text: French)

Competition between structural steelwork and reinforced concrete, which began 60 years ago, grew more severe with the introduction of new techniques of prestressing and the appearance on the market of very high tensile steel reinforcement. Not only did the percentage of steel used in building diminish, but the proportion of bridges and structures built in steel also fell off.

The recent appearance of a new type of binder capable of giving concrete a greater resistance to compression and a capacity to withstand tensile stress, could make this competition even more severe for the fabricators of steel structures whose interests are also being affected by the increased use of timber.

The steelmakers could not remain indifferent to these limitations on the use of a range of products which form an important part of their production. They have retaliated by putting on the market plates and sections in high yield stress steel. The European steel producers first extended their range to steel 42 and then to steel 52, but have gone no further. The Americans on the other hand offer a steel grade 56 and even of grade 80 in the case of T.1 steel.

It is felt, however, that the range of application of these steels with very high yield stress must be rather narrow as their employment is restricted by the limitations imposed by such factors as deflection and fatigue.

It is well-known, that the modulus of elasticity of all steels is practically constant. Even in mild steel structures, the designer is often unable to exploit all the resources offered from the stress point-of-view because the resulting deflections would not be compatible with user requirements. Nothing can be gained, therefore, by further extending these resources.

It is also well known that the question of fatigue is of great importance in structures. Resistance to fatigue rapidly diminishes as the fluctuation of stress increases. Resistance to fatigue is also affected by the existence of internal stresses in all rolled sections and particularly in joists.

It was necessary therefore to overcome these obstacles by finding a method, which also opened the way to a much wider use of high tensile steels, which are able to resist the competition of other present-day materials. Such a method is the preflexion technique. Beams fabricated by this process are known on the market as PREFLEX beams.

Invented in 1950 by a consulting engineer, Mr. A. Lipski, and developed in close collaboration with the late Professor L. Baes, the preflexion process is in wide use, finding more and more applications even in fields where steelwork had been eclipsed, such as bridges of small and medium span, by amalgamation of steel and concrete.

Those who wish to widen their knowledge of the process are referred to the abundant literature on the subject, a selection from which is given in the annexed bibliography.

Let it be said that the preflexing technique, which comprises a special encasement of high quality concrete, makes it possible:

(1) to reduce deflections to about one-third of the usual value;

(2) to reduce fatigue effects to a considerable extent. (A beam tested in the Otto Graf Institute in Stuttgart withstood two million reversals of stress at the calculated working load, then two million at working load + 25% and a further million at working load + 50%);

(3) to avoid difficulties arising from internal stresses;

(4) to remove the combination of unknown factors which, in normal construction, necessitates the use of a large load factor;

(5) to ensure constant stiffness and an unvarying conservation of material.

(6) to exploit the high qualities of steel up to 80% of the yield stress, thus achieving appreciable economies, not least of which is the reduction of dead load.

The Preflex System has made it possible to construct an appreciable number of bridges, viaducts and tunnels with openings of from 10 to 50 m (33 to 164 ft.) with construction depths reduced to 1/45 of the span and using beams the dead weight of which, including encasement, represents only 1/35 of the total load.

The general details are given in the illustrations.

The use of Preflex beams is particularly economical for structures where headroom is restricted, such as elevated highways, and intersections of motorways and railways.

When considering flyover and river crossings, it should be noted that no scaffolding is needed for Preflex construction.

Although manufactured in Belgium, France, Great Britain, Holland and South Africa, the preflexing technique also offers new export possibilities, since Preflex beams fabricated in Europe have already been supplied to countries as far-flung as the Ivory Coast and Ceylon.

Needless to say, the preflexing process is available to all who design in structural steelwork.

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10 Ir. Ch. Z. Zollmann: Acero Estructural Preflexado — Construcción — Agosto de 1961. New York.

Description of photographs

1 — Transportation of a 50.30 m. long Preflex steel girder for bridge on Antwerp-Liège highway.

2 — Railway bridge at Boirs (Antwerp-Liège highway), two 28 m. long spans.
Slenderness: 1/25.

3 — Elevated crossing over railway track Paris-Marseilles, constructed without hindrance to traffic.

4 — 200 m. long bridge spanning the Cheratte section of the King Baudoin highway, constructed with prefabricated slabs together with Preflex girders.

5 — Railway bridge over the Djili at Matadi.

6 — Urban viaduct. (Brussels)
Slenderness: 1/42.

7 — Upstream bridge of the Vianden hydraulic power-station. Three and four continuous 25 m. spans.

8 — Bridge (leg-frame supports) over the Demer at Aerschot.
Slenderness: 1/38.

9 — Bridge at Merksem.
Slenderness: 1/32.

10 — Bridge over the Slotervaart at Amsterdam.
Slenderness: 1/36.

11 — Bridge in West Norfolk. (England)
Three spans of 18 m. each.


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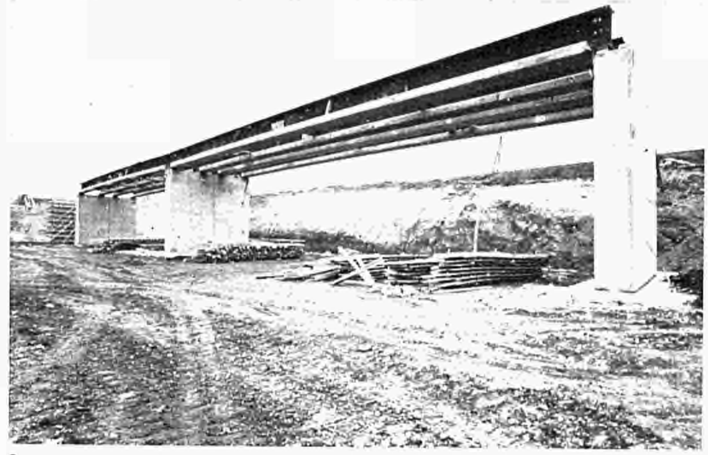
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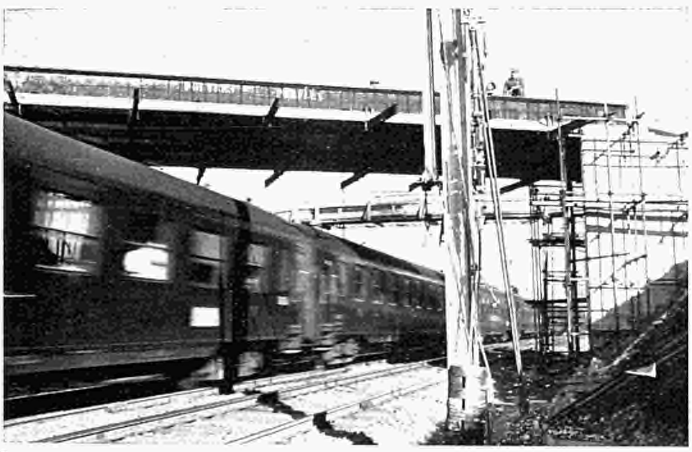
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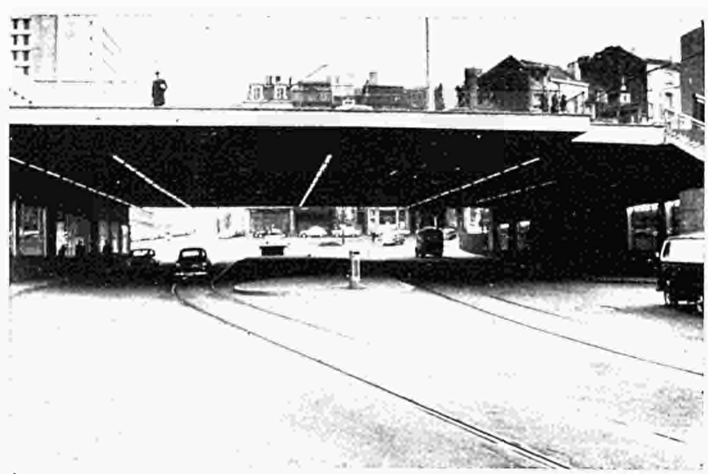
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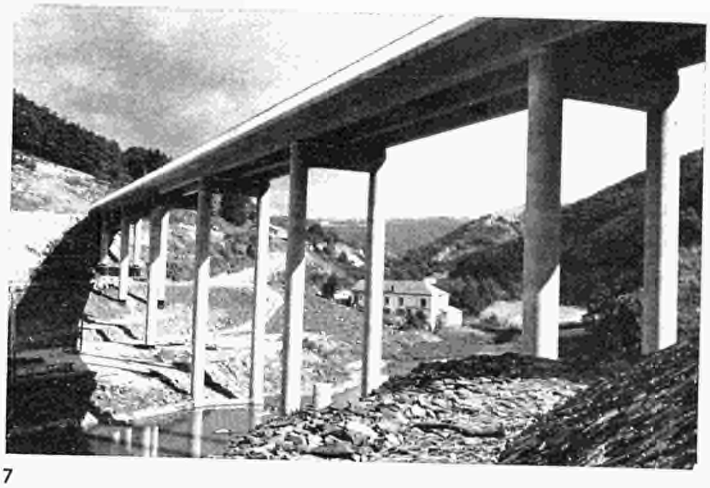
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Jean-André RORET

The Situation with regard to Steel Bridges in France

(Original text: French)

I should like briefly to outline the present situation with regard to steel bridges in France, which is directly affected by competition with two other materials, reinforced and prestressed concrete. Clearly large spans are favourable to structural steelwork. We have seen in France that for spans of above 70 metres, concrete cannot compete with steel. However, for lesser spans a battle is in progress.

Our weapon, although it is not an absolute one, is high tensile strength steel, which, in France, has been standardised and is known as Steel 52 with a guaranteed yield point of 36 kg/mm². The permissible stress under the French Bridges and Highway regulations is 27 kg/mm² for tensile and 24 kg/mm² for compressive stresses. There is no question of our fighting prestressed concrete with ordinary 42 kg mild steel with a guaranteed yield point of 24 or 26 kg/mm². Moreover, where the main part of a structure is in Steel 52, some of the secondary members, such as struts and certain types of stiffeners can still be made of Steel 42 for the sake of economy.

The other weapon at the disposal of the steel constructor is of course welding. It is at present impossible to conceive of a mainly high yield stress structure as being other than welded. It could of course be argued that every French bridge, and every bridge abroad, has parts welded on the site and the problem is to know whether welding must be done on site or whether other assembly methods can be employed. You see, for the past twenty years the French practice has been solely on-site welding. This has obviously not made us neglect other assembly methods on some projects, especially those using high tensile strength bolts.

I believe that we must decide which new weapons or improvements in our present weapons we must use to ensure greater success in the utilization of steel.

One possible line of improvement is the use of steel with mechanical properties even superior to those of the steel which we are now considering, i.e. with a tensile strength of 36 kg./mm². We have carried out systematic tests on steel with a tensile strength of 45 kg./mm²., and we found that by applying a little more preheating treatment, something we did not do with steel 52, the precautions we took with the welding of our high-tensile strength steel are adequate to carry out welding operations on 45 kg steel quite safely. We also made welding tests on steels with even higher tensile strength, including steels similar to the American A.C.T.I. with a tensile strength of 80 kg./mm². I must say that as an engineer I reserve my judgment on the large-scale industrial welding of these steels. I believe that above 40 or 50 kg./mm². we can no longer regard it as a welding operation, but a metallurgical laboratory operation which has to be made anew each time, and I consider it rather hazardous. This is my opinion today, and I should be only too pleased to be able to alter it as soon as possible. We builders, architects, financiers and government engineers must give free rein to our imagination and show more courage in adopting new techniques on the lines which some of our predecessors have shown us, or on the lines which today appear likely to lead to exciting solutions.

The other main improvement may well be brought about by the large-scale use to increase the strength of the structure as a whole, of reinforced concrete slabs, which do of course generally have to be employed in the upper parts of our bridges. There are certainly a number of problems — shrinkage of concrete, the permissibility of stress in reinforced concrete. These problems can be found. Yesterday, we saw the brilliant solution conceived by the Swiss for the Sarine bridge with prefabricated members. In my opinion, almost all these techniques will be based at any rate on longitudinal prestressing, sometimes backed by transverse prestressing to ensure the best bond between the concrete slab and the steel girders.

I believe also that a great deal of progress can be made in the use of high-strength bolts. The regulations are not very consistent, and I believe that a series of systematic tests such as those now being conducted by the European Convention on Steel Construction Work will allow us to make further progress in this field. Due attention must also be given to improvements in connecting members, such as support stanchions, widespread use of neoprene when it is solely a matter of pivot or swivel supports, and the addition of teflon when rotation is accompanied by slip. I believe that in general we must strive to optimise the cost of the construction. To those who listened to my talk in Brussels a few months ago, I said the cost of a bridge was the total cost of the foundation plus that of the superstructure. It is no good having one of the two factors lower than the other, or having a very low total cost while the cost of the superstructure is only the product of weight times unit cost, for I believe that systematic research for ultra-light weight means nothing if the total weight multiplied by unit cost is not the lowest possible.

Roberto BONAMICO

New Steel Applications for Composite Steel-And-Concrete Bridge Structures

(*Original text: Italian*)

For some time now, both inside and outside the Community, steel has been used in composite steel-and-concrete bridge structures by combining a reinforced concrete slab with steel girders. In Italy the system is known by its German name of *Verbund*. The slab distributes the loading laterally and takes up the compression forces longitudinally, whilst the steel girders absorb the shear and tensile stresses.

The advantages of the system are well known:

(a) elimination of centring;

(b) less shuttering;

(c) a reduction in dead weight;

(d) better steel utilization. The reduction in weight is greater on the central span than at the sides.

If we take a beam A.B. simply supported at the ends Fig. 1) subject to a load p uniformly distributed over the entire span 1, the bending moment M_c acting on the centre-line C will, as we already know, be as follows:

1.1 $M_c = 0.125{,}00\ pl^2$

If (Fig. 2) the uniformly distributed load p were to relate to the centre half (DCE) only, the bending moment

M'_c acting on the centre-line C would be:

1.2 $M'_c = 0.093{,}75\ pl^2 = 0.75\ M_c$

and if (see figure 3) it were to relate to the two side quarters (AD and EB) only, then it would be:

1.3 $M''_c = 0.031{,}25\ pl^2 = 0.25\ M_c$.

In other words, reducing the load on the centre half reduces the bending moment on the centre-line C by three times as much as reducing the load on the two side quarters by the same amount.

Let us now (Fig. 4) take the same beam AB subject to a uniformly distributed load 4 p, four times greater than the previous load, on the two side quarters, 1.3 becomes:

1.3.1 $M'''_c = 4\ M''_c = 1\ M_c$

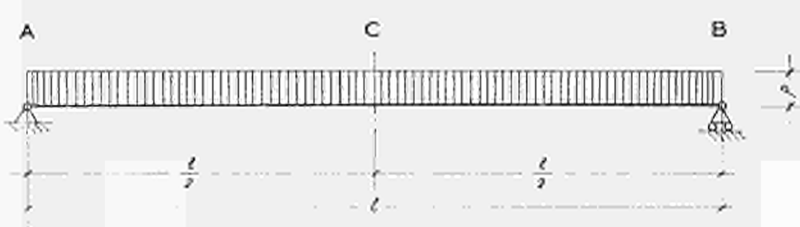


Fig. 1

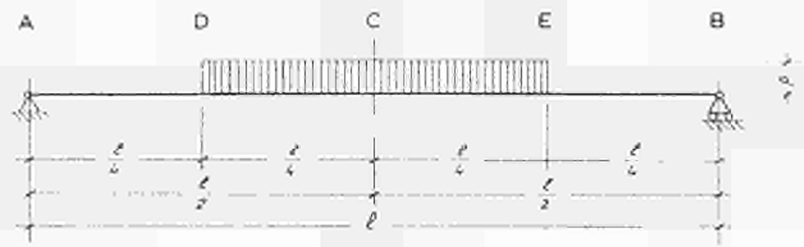


Fig. 2

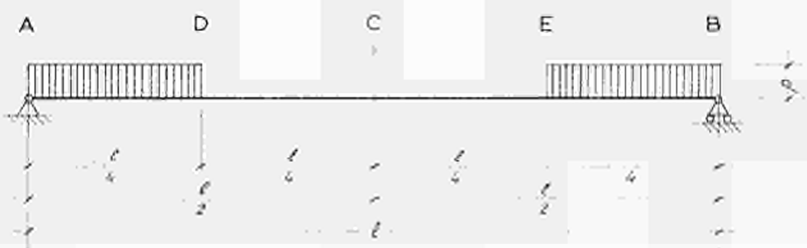


Fig. 3

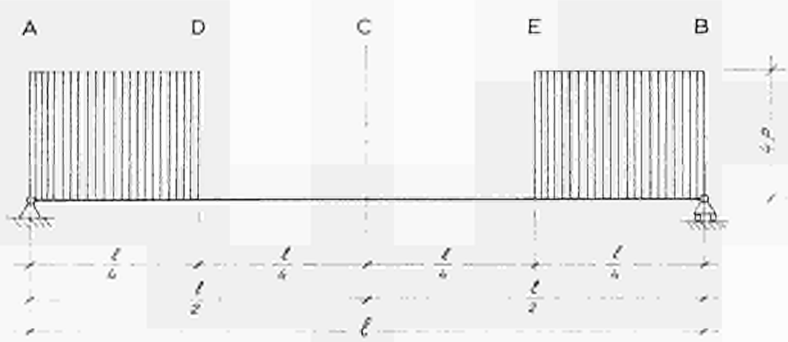


Fig. 4

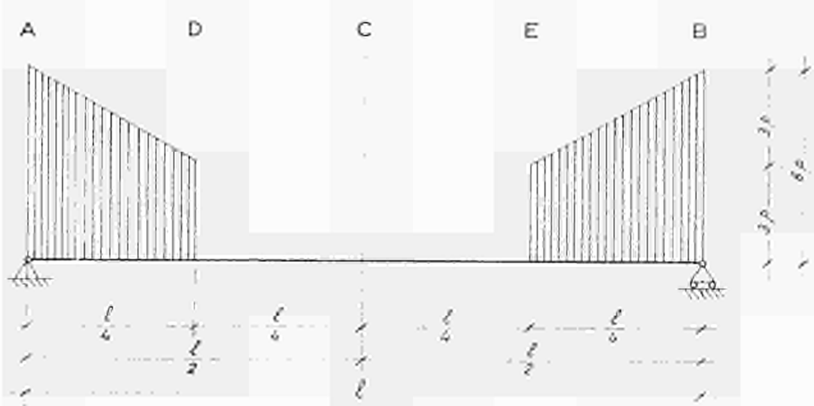


Fig. 5

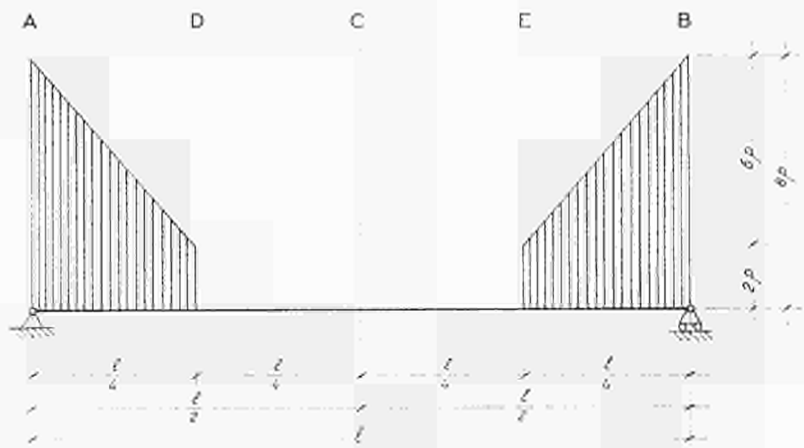


Fig. 6

i.e. the bending moment exercised on the centre-line by a load uniformly distributed over the whole span equals that exercised on the centre-line by a uniformly distributed load four times as great over the side quarters only.

The same bending moment M_c on the centre-line C is achieved with trapezoid loads on the side quarters varying from 3 p to 6 p (Fig. 5) or 2 p to 8 p (Fig. 6).

In the case of parabolic trapezoids (Fig. 7), *i.e.* a parabolic triangle plus a rectilinear trapezoid, equal values (*e.g.* 2 p) on the two quarters (D and E) give higher values (10 p) at the two ends (A and B) with an equal bending moment M_c on the centre-line C.

We find, for example, that the bending moments induced by the dead weight of a concrete slab 20 cm thick spanning the entire width and by that of a slab 80 cm thick limited to the side quarters are identical.

If the slab thickness varies instead of being constant e.g. increasing linearly from the quarters towards the supports, cantilevers from 0.60 to 1.20 m or from 0.40 to 1.60 m are possible with an equal bending moment M_c on the centre-line C. Generally speaking, every centimetre of thickness on the quarters will provide 2 cm extra thickness at the ends.

If we now introduce a parabolic instead of a linear variation, the phenomenon increases in proportion to the degree of parabola. With the second degree, every centimetre of thickness lost on the quarters provides 5 cm extra thickness at the ends. A slab varying from 0.40 to 2.00 m, for example, would be represented by figure 7.

These calculations led me to design a steel-and-concrete structure in which the Verbund combination of two materials takes place, not in a horizontal section, but in two vertical sections symmetrical to the centre-line of the span, with the

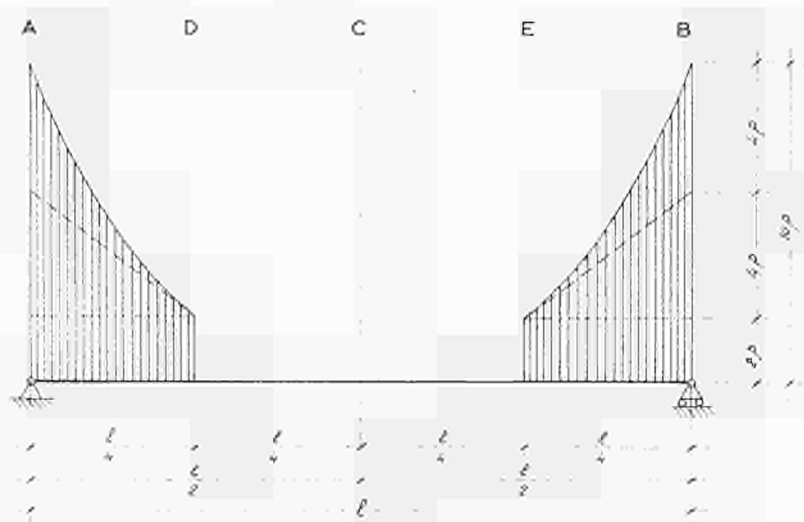


Fig. 7

central portion entirely of concrete. The trapezoid shape of the side portions, although not essential, helps in subdividing the total bending moment between the centre-line section and the two end sections.

As regards the apportionment of total bending moment between the centre-line section and the end sections, reference should be made to an earlier article of mine in Autostrade (No. 12, 1960) entitled Alcune considerazioni sull' impiego delle strutture a telaio (The use of frame structures).

On that occasion my aim was to establish the optimum apportionment of the bending moment exercised by a load uniformly distributed over the entire span, and for purposes of comparison I utilized the area of the diagram of absolute bending moment values.

Taking a Cartesian diagram (figure 8), with the ratio λ between the reciprocal distance of the nil points and the total span as abscissae and the ratio $f(\lambda)$ between the corresponding value of the area of the diagram of absolute bending moment values and the value of such area for $\lambda = 0$ as ordinates, we find that the function $f(\lambda)$:

(i) has a minimum (0.75) for $\lambda = 0.5$;

(ii) is ≤ 1 for $0 \leq \lambda \leq 0.75$;

(iii) rises rapidly from 1 to 2 when λ rises from 0.75 to 1.00.

In other words, the area of the diagram of absolute bending moment values:

(i) is least when the zero points are on the quarters of the span and is only 37.5% of the value for a beam simply supported at its ends;

(ii) does not exceed 50% of the value for a beam simply supported at its ends if the zero points are more than 1/8th of the span away from the ends;

(iii) rises rapidly from 50% to 100% of the value for a beam simply supported at its ends if the zero points are less than 1/8th of the span away from the ends.

Under optimum conditions, the total bending moment is distributed between the centre-line and the ends in the ratio of 1:3.

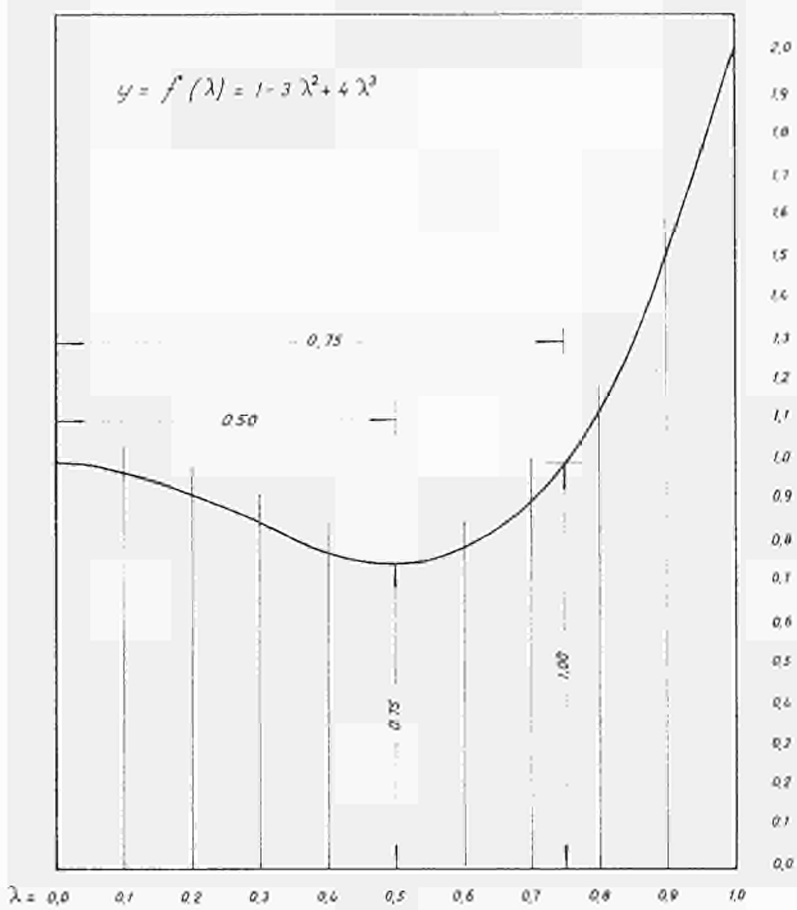


Fig. 8

Without going so far as to regard the quantity concerned, namely the area of the diagram of absolute bending moment values, as an absolute parameter, it seemed to me that it might provide some idea as to the possible variation in load-bearing material needed for the structure, and thus in cost. Moreover, the curve shown in figure 8 reveals that around the optimum condition the variation in this area is slight, which means that even if the parameter were exact, abscissae values close to the optimum would only engender slight variations in the ordinate.

I therefore decided to distribute the entire bending moment between the centre-line and the ends roughly in the ratio of 1:3 in a composite structure. The central span is of steel and the two side spans of concrete. The joint section is close to the zero point, so roughly speaking the all-steel part covers the centre half and the concrete portions the two side quarters.

An example of my design (Fig. 9) is a three-span bridge 18.00 + 47.60 + 18.00 m whose central portion (21.20 m.) consists of two IPE/400 girders weighing 368 kg/sq.m placed side by side and joined by welding the flanges together. The side portions are made up of reinforced-concrete slabs 13.20 m long varying in thickness between 0.48 and 1.60 m.

In the 47.60 m. central span the dead-weight bending moment totals 226 metric tons/m. and therefore corresponds to a uniformly distributed load of 798 kg/sq.m., *i.e.* a uniform concrete thickness of 0.319 m. The maximum stresses are 1,800 kg./sq.cm. in the central steel portion and 100 and 1,400 kg./sq.m. at the sides in the concrete and reinforcement respectively. The permanent static load is 250 kg./sq.m. and the live load 823 kg./sq.m.

The quantity of steel required for a superstructure width of 9.50 m. is 169.3 metric tons made up of 75.2 metric tons of IPE/400 structural sections and 94.1 metric tons of steel reinforcement bars.

An approximate comparison between the materials used in my design (A) and those normally employed in the best and most recent Verbund types (B) gives the following figures per sq.metre of superstructure surface:

	A	B
1. Structural steel	94 kg.	128 kg.
2. Reinforcement bars	118 kg.	30 kg.
3. Total steel	212 kg.	158 kg.
4. Concrete	0.68 cu.m.	0.27 cu.m.
5. Shuttering	1.04 sq.m.	1.23 sq.m.

Although version A uses more steel and concrete, it should not work out dearer than the composite version B. The structural steel assembly costs are limited to welding together the girders and painting (virtually only the underneath). 200 cm of weld, 100 cm on top and 100 cm underneath, are required to join every metre's length of girder (i.e. 66.3 kg) to the one before it, making 3 cm/kg. Painting is virtually confined to the underneath of the sections, *i.e.* 1.800 sq.cm. per metre's length of girder, making 27 sq.cm/kg. There is no need to paint the internal surfaces of the girders, since they are sealed, nor the top surface, which is protected by the decking. Allowing for the external surfaces of the two end girders of the box, paintwork amounts to 30 sq.cm./kg. The estimated cost of these two operations — welding and painting — is about 20% of the price of IPE/400 girders as delivered. Allowing an extra 5% for erection, this makes a total increase of 25%. In view of the simplicity of the concrete reinforcement, this increase can be regarded as valid for the latter as well. Since reinforcing bars cost only 64% of the price of IPE/400 girders (bars 55.6%, girders 44.4%), the resulting total erected steel cost equals that of the IPE/400 girders prior to erection. The cost of structural steel processing usually results in at least doubling the cost of the unerected material, so a 34.3% increase in the quantity of material will still result in a saving of at least 32.9%. Moreover, the greater volume of concrete not only requires less shuttering but is thicker, so that placing costs are lower.

A prototype of my design A is at present being built for the Cassa per il Mezzogiorno (Southern Italy Develompent Fund) at Monte San Giovanni Campano in the province of Frosinone, about 100 kms. from Rome. This is a four-span viaduct 16.50 + 2 × 22.00 + 16.50 m., making a total length of 77.00 m. The 22.00 m. symmetrical central spans amount to slightly less than half the central span of design A (47.60 m.), and IPE/200 girders are used instead of /400.

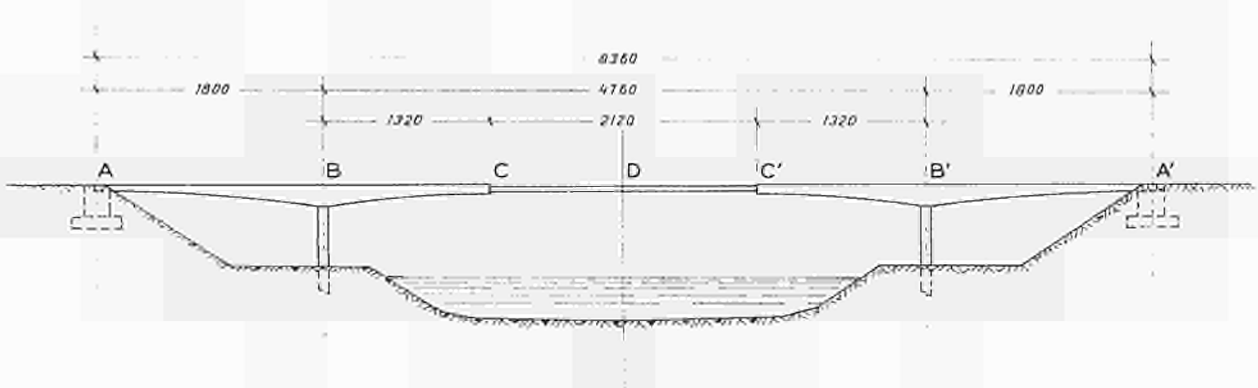


Fig. 9

My design is not intended as a substitute for the *Verbund*, which has been so successfully employed in Italy in recent years, mainly due to the efforts of Costruzioni Metalliche Finsider (C.M.F.) of Milan, but it does enable large quantities of steel to be used, particularly in the Euronorm range of structural sections, by firms still relatively unfamiliar with constructional steelwork. This is the case with the majority of Italian building companies, whose gradual development along these lines will thus be encouraged.

The central steel portion can be regarded as a wedge gradually penetrating the structure and reducing the volume of concrete by pushing it towards the supports. The depth of penetration will largely depend on economic competitiveness against concrete or steel structures, which can certainly be guaranteed to an increasing extent with high-strength steels as and when future regulations allow their special strength properties to be exploited.

Fabrizio de Miranda

(Original text: Italian)

With reference to Prof. Kihara's admirable slides of Tokyo's new structural-steel elevated roads. I should like to ask him what criteria were adopted in the very early planning stages for selecting the type of structure, the size of the standard spans, and the type of pillars.

As all bridge designers know, once you have the information on the road, the longitudinal section and the subsoil structure and know the basic data of the project, the main aim usually is to see that the job is done at minimum cost. Along with all other considerations of technology and pleasing appearance, you have to bear in mind the importance of choosing the right statical plan and the disposition of the load-bearing factors in such a way as to keep the overall cost of the structure to a minimum.

The problem is not a new one, but nowadays it must be tackled differently than in former years, especially with regard to steel structures.

In the past, as far as I know, designers very often (though not invariably) dealt with it more or less "at sight," basing their ideas on previous experience and gauging from rough approximate calculations what might on the face of it be the most suitable mode of approach.

The preparatory plans considered were nearly always a mere handful, whereas in fact — and particularly, I repeat, in the case of steel structures — the number of possible statiscal plans and possible ways of building crossovers and flyovers is enormous, very much larger than for other types of construction. The trained engineer has little difficulty in picking a number of types, but he will find it a real problem to choose the structure, so he will go by intuition — which is worth quite a lot, but not, nowadays, enough.

The trickiest part, in this first stage of the planning, lies in drawing up the cost charts; though this is not really difficult, provided job-card account figures and other data are up-to-date and reliable. By this means, which does definitely require precise and accurate computation (but the initial work is carried out quickly by machine), it is possible to obtain the cost curves for various statical and load-distribution plans. Attention is then concentrated on those of the more satisfactory possiblities which may be considered the most suitable on grounds of technological practice.

Such a study in itself usually makes it possible to work out arrangements averaging about 10% lower costs than those based purely on intuition, experience and rough initial calculations.

May I ask what Prof. Kihara has to say on this point?

Azarius Dobruszkes

(Original text: French)

Professor Kihara, could you tell us what precautions are taken in welding steels with a very high yield point? Also, what is the range of shapes available: only flats, or do you also have rolled sections, and in particular I-beams?

Hiroshi KIHARA

(Original text: English)

First of all, I will explain very briefly what is meant by preliminary design and design formula. The selection of the type of bridge, whether concrete or steel, is such a complicated matter that it is very difficult to explain it in a few words. It depends partly on who places the orders, whether Governments, local authorities or public corporations, and generally the economic viewpoint is very important. In Japan, the price of a bridge is composed of the sum of the costs of the superstructure and substructure; therefore steel bridges of long span are more widely used than concrete bridges, and this tendency is increased by the fact that confidence in the safety of steel bridges is greater than in concrete bridges.

Loadings, allowable stresses and principles of detail design, etc., are laid down in the Design Specification for Steel Highway Bridges or National Railway Bridges, as they are in A.A.S.H.O. in the U.S.A., B.S.S. in the U.K., D.I.N. in Germany, etc., and we have many standards not only for the detail designs for various types of steel- and concrete bridges, but also for welding procedures.

Allowable stress is calculated by approximately Y/1.7 for steel S.S.41 and S.M.50 and Y/1.8-1.9 for H.W.50.

Minimum weight design is easily calculated by computer, but minimum cost is generally the most important consideration. Consequently, it should be decided by considering the price of steel (mild steel and high strength steel), labour charges and wages, transportation charges, and so on, which would depend on the conditions in the country concerned.

I would like to say a word about prefabricated bridges, which are not dealt with in my paper.

In my personal opinion, there are two kinds of prefabricated bridges in Japan. The first is the temporary bridge, used for emergencies, for instance, when a bridge has been carried away by a typhoon. We call this type a "Panel Bridge," which is a kind of modified U.S. Army bridge.

The second type is a bridge of short span (8-18 m) known as "Prefabricated Standard Bridge," having a broad-flange H-shape main girder. After this H-shape main girder has been cut to the specified length, gusset plates and slab anchors are welded. When full-length girders cannot be transported, main girders can be delivered in short pieces with standardized joints, which can be readily assembled on the site by high strength bolts. Major parts of cross beams are channels, which are fastened to the main girders with 3 high strength bolts. Before the main girders are erected, several kinds of shoes (for instance synthetic rubber shoes) bre placed by the specified methods. The main girders and cross beams are generally test-assembled before shipment. aut many units of the same length are comprised in one shipment; the preshipment assembly and tests are conducted on a single typical unit. In the assembling of the main girder-joints on the site, the doubling plates are attached and high strength bolts are used. After the main girders have been erected, cross beams can be assembled easily by bolting them to the main girder gusset plates. After the assembly of main girders and cross beams is completed, floor slab moulds are assembled and floor slab reinforcing bars are placed in position.

Concerning „weldability," I would like to make the following two observations. The first problem is how to avoid welding defects in welding procedure, and the second how to ensure safety in the use of welded structures. Under the first head, the most important point is the welding crack, which will be tested by the "Tekken Restraint Cracking Test" (Y-type groove); under the second, the material needs not only notch toughness for the prevention of brittle fracture at low stress level, but also ductility in the welded joint for external loads, tested by the so-called "Austrian Test" (bead bond test). These cracking and bead-bending tests are carried out at the appropriate preheating temperature (for instance, lower than 75° C for H.W.50, and 150° C for H.W.70). Generally speaking, the results of these cracking and bead bend tests have very good correlation with the maximum hardness at the heat affected zone (HAZ), where cold cracks and bending cracks would be able to start, especially in high tensile steels. Consequently, the "Standard Maximum Hardness Test of I.I.W." (International Institute of Welding), in which welding is done at room temperature without preheating, can generally be used in place of the two abovementioned tests; also, the carbon equivalent (C.eq.) for the maximum hardness can be easily obtained from the chemical compositions. As a matter of fact, when a high tensile steel approved by the Japan W.E.S. is used for a bridge, it is sufficient simply to check whether the C.eq. of the steel satisfies the values shown in Table 3 of my report: if not, the standard maximum hardness test should be carried out.

Where the maximum hardness thus obtained exceeds the values shown in Table 3, the bead bend test should be carried out to assure that the surface strain, before crack initiation, exceeds the required value of 23%.

M. Demol

(Original text: French)

The E.C.S.C. has for some years been engaged in a major drive to secure the rationalization and standardization of rolled products, and more especially of beams and joists. May I ask M. Dobruszkes whether he considers the product now on the market satisfactory as regards size? The IPE and broad-flanged series are now available; does he think a study ought to be made of sections of different size?

Azarius Dobruszkes

(Original text: French)

The question is very much to the point, since in fact in Europe we have a smaller choice of sections than, say, in America. We would like to have sections with the largest possible flange width, as much as 400 or even 450 mm., such as are made by United States Steel and Bethlehem Steel.

Findings

The very interesting reports presented to the Committee on bridges and allied structures showed the trend in the use of steel for the construction of these very important structures.

The emplasis was on the use of steels having a higher yield point and, above all, weldability.

The weldability of these high-strength steels on site is still associated with some uncertainties.

Welding is extensively employed for parts prepared in the fabrication works, where this operation can be carried out with the greatest possible care and with all necessary supervision and inspection to ensure success. For the erection of structural components on site, the methods almost exclusively applied in many countries are riveting or high-strength bolting.

As a result of the use of steels with a higher yield point in the more heavily loaded parts, the structure becomes more economical and competitive — also in cases where other materials appeared to be at an advantage.

The use of elevated roads, sometimes two or more placed one above the other, has provided the solution to the traffic problems where the presence of existing obstacles ruled out the possibility of increasing the capacity of the traffic routes.

It was clearly shown how desirable and useful it is, in order to achieve a further increase in the competitiveness of steel, to get away from the conventional shapes of the rolled sections that are commercially supplied and to develop new shapes by using easily weldable steel sheet which can be formed to the most rational shapes, more efficiently able to resist the loads and resulting in a reduction in the weight of material used, while at the same time possessing greater rigidity as a result of taking maximum advantage of the high modulus of elasticity.

In this domain a very interesting development is the stainless steel girder of quite novel shape produced by the welding of sheet.

Steel bridge construction already has a long history, but up to the present time steel has been considered a substitute for other materials, and its properties have not been utilized to the full.

Hence it is desirable that those who engage in steel construction should be better informed of the possibilities of this material, so as to derive maximum benefit therefrom.

WORKING PARTY II :

Roads and Roadway Accessories

Chairman:

Prof. Dr. techn. Friedrich REINITZHUBER

Rapporteurs:

Dipl.-Ing. Heribert THUL

Dr.-Ing. Helmuth ODENHAUSEN

Dr.-Ing. Saverio SCHULTHEIS BRANDI

The Working Party dealt with present and possible future uses of steel for various purposes connected with roadmaking :

(a) roadmaking proper (use of steel to reinforce the decking, permanent steel roads, steel connecting components);

(b) temporary steel roads (level or elevated);

(c) roadway accessories (footbridges, culverts);

(d) road safety devices, road signs and lighting.

With regard to (b) and (c), the Working Party discussed the advantages to road users of quick installation; it was agreed that this factor must be taken into account in comparing the respective economic advantages of steel and other materials for these purposes.

More generally speaking, it was noted that roadmaking was a comparatively new field for steel : consequently, while much research and experimentation are still needed, steel was felt to have prospects of expansion in this connection.

Heribert THUL

The Road System in the European Economic Community

(Original language: German)

During the last two decades, traffic on all West European roads has increased both in extent and in weight. Great attention has therefore had to be paid to the building of new and the extension of existing roads, as well as to the maintenance of the latter. Economic interconnections reaching across the frontiers of States also necessitated supra-regional planning and road building, as is now made very evident, for example, in the "European Highways."

The present overall length of the roads in the E.E.C. area amounts to about 1,452,000 km.

These roads are subdivided into approximately :

5,300 km. of motorways,

687,200 km. of other classified roads and

759,500 km. of unclassified roads.

At the end of 1962, about 20,700 km. of the total network were classified as "European Highways."

Table I gives the figures broken down into countries. It shows that West Germany occupies first place where the network of motor-roads is concerned, but that France has by far the biggest network of roads.

Table I — Road network in E.E.C. countries. Latest figures available. (in km)

Country 1	Motorways 2	Other classified 3	Unclassified roads 4	Total length 5	European Highways 6
Belgium	200	22,900	70,000	93,100	1,101
West Germany	3,077*	148,807*	247,531**	399,415	5,792*
France	243	340,000	380,000	720,243	5,943
Italy	1,135	160,173	35,000	196,308	6,427
Luxembourg	—	2,095	2,165	4,260	90
Netherlands	4 lane 189 2 lane 436	13,170	24,800	38,595	1,345
Total	5,280	687,145	759,496	1,451,921	20,698

* As at 1.1.1964.
** As at 1.1.1961.

The traffic density on the E.E.C. roads is given in Table II. Here we see that West Germany, by comparison with other West European countries, lags behind in motorization (based on the number of inhabitants), but that the traffic loads of our highways are extremely high. It is not to be expected that there will be any change in this position during the next few years, as the annual rate of increase in the number of motor vehicles is about 10%. It is therefore vital to promote the building of new roads and also to maintain existing ones to such an extent as will enable us to keep pace with this increasing volume of traffic. The problem is not completely solved, however, merely by increasing the means for road building : construction processes must be improved, and means found for building roads rapidly and smoothly without appreciably impeding the traffic.

Table II

Number of motor vehicles

Country	Per 1 km. of road *	Per 1000 inhabitants **
Belgium	12	96
West Germany	20	89
France	14	126
Italy	23	42
Luxembourg	19	132
Netherlands	20	57

* Position 1959.
** Position 1961.

Possible means of avoidin traffic jams on motor-roads

Disturbances to the flow of traffic amount to losses to the national economy. During the last few years considerable traffic jams have been observed, particularly at points where the West German motor-roads were being rebuilt or converted, so that it was necessary to adopt every possible means of overcoming this evil — an evil which was aggravated year by year. It is no longer possible as it was in former times, when the traffic density was very slight — to close a section of the highway while alterations are in progress; every praticable step has to be taken to ensure that the interruptions caused by road building operations are kept down to a minimum. In selecting the measures required for this purpose, consideration has naturally to be given to economic factors, having regard not only for the national economy as a whole but also for the welfare of the individual.

In this paper we shall deal only with the problem of traffic-jams on our motorways, because these jams that have a far-reaching effect on the traffic situation, and it is around this factor that the use of temporary steel roads must centre.

Each lane on a motorway can cope with, as a maximum, 1200-1500 vehicles per hour. The capacity of a lane cannot be increased by more traffic regulations. If, therefore, more vehicles are to pass over a lane during a given period, traffic jams must inevitably occur. For this reason, additional measures to maintain the four-lane traffic at the reconstruction points have become indispensable. The possibilities available are as follows :

(a) diversions;

(b) the provision of temporary overhead roads;

(c) the provision of temporary steel roads;

(d) re-subdivision of the lanes into narrower auxiliary lanes — if necessary, with the inclusion of waiting lanes, lay-bys or hard shoulders (the "Stud" solution).

No special explanatory remarks are required in connection with the *diversions*. Unfortunately, they frequently cannot be carried out — at all events, not economically. In many cases, moreover, a diversion is merely a transference of the traffic jam from one road to another.

These circumstances suggested the desirability of temporary overhead roads, particularly since modern erection methods made it possible to erect thoroughfares of this kind quickly and without any appreciable disturbance of traffic. We have gone into this problem very thoroughly and, in co-operation with various German firms, have designed overhead roads and made an accurate study of their erection and removal. Tempting though such a solution appears at first sight, it was nevertheless not only very costly but, above all, it had an element of danger, since the traffic has to be transferred to the 2nd and 3rd level (Fig. 1).

It was therefore a great step forward when, in the autumn of last year my predecessor, Dr. Klingenberg, suggested that the steel roadway should be transferred simply to that part of the highway which was unaffected by the reconstruction operations, and not to the 2nd or 3rd levels. The slab-width in this scheme was selected specifically to provide for four-lane traffic on the transferable steel road.

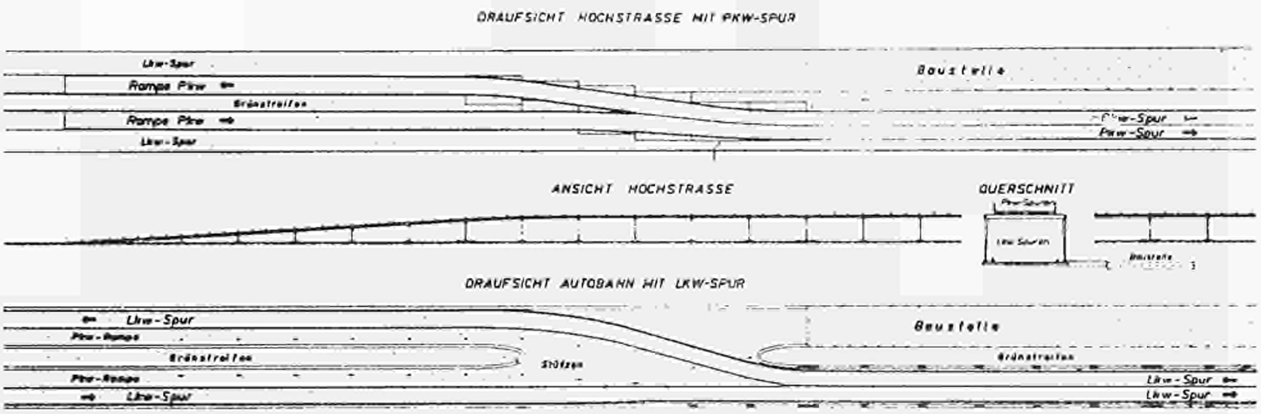


Fig. 1

The firm of Krupp conceived this idea at the same time, and it seemed therefore obvious that the development of the temporary road to the stage where it could be applied in practice should be placed jointly in the hands of Krupp and the West German Transport Ministry. But it was not least due to Dr. Seebohm the Transport Minister, who very quickly realized the advantages which this solution offered by comparison with the overhead-road solution mentioned above and who showed such courage in reaching an early decision, that the first steel "level" road could already be made available to traffic in the spring of 1964.

Figure 2 shows diagramatically the solution based on flat steel roads. It can be seen that four-lane traffic is possible without hindering the roadbuilding operations, there being no obstruction to the flow of traffic.

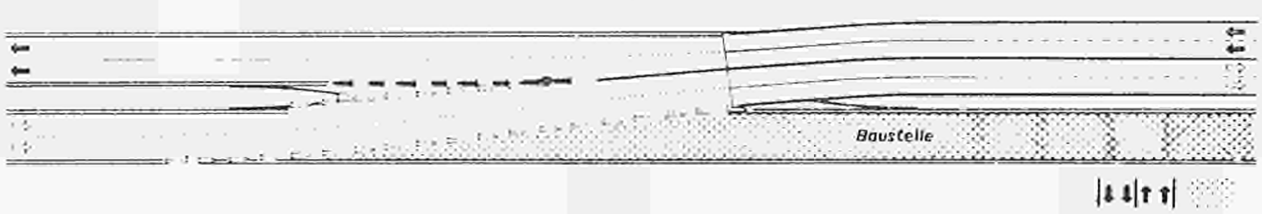


Fig. 2

From the flat steel road to the "stud solution" already mentioned was no very great step. The usual cross section of our motor-roads, with two lanes of 7.50 m. each in width, a 4 m. wide central strip, and four side strips of 0.75 m., make it feasible to arrange a number of emergency lanes on a road, instead of the two existing lanes of 3.75 m. width. The emergency strips must naturally be made narrower and we have planned for a width of 2.50 m. for private cars and 3 m. for the remaining traffic. The reduction in the widths, however, necessitates corresponding limitations of speed, in view of possible collisions. The boundaries of the lanes are marked by "studs" (plastic discs) attached by adhesives.

The stud solution offers the following possibilities :

(a) The provision of three emergency lanes on that part of the road which is unaffected by the conversion operations and of a fourth emergency lane on the road adjacent to the working site (Fig. 3), or

(b) The provision of four emergency lanes on that part of the road which is unaffected by the conversion operations, if a "lay-by" or "hard shoulder" is available which can be utilized as a driving lane (Fig. 4).

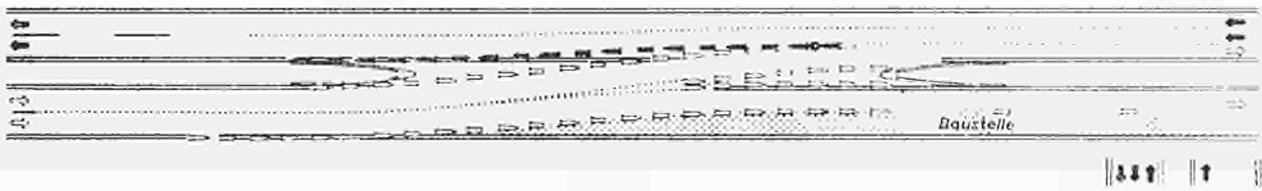


Fig. 3

Fig. 4

Four emergency lanes can also be accommodated by the inclusion of the median strip of the motorway, if this is to be renewed in any case.

Figure 3 shows how the provision of a fourth emergency lane on the road by the side of the working site reduces the working space to about half the width of the normal roadway. If the road-bed has to be reconstructed to an excessive depth, this solution is unsuitable, owing to the danger of the subsidence of the fourth emergency strip. On the other hand, figure 4 shows that if hard shoulders or lay-bys are provided at the side, satisfactory four-lane traffic can be organized during the conversion operations. Unfortunately, however, only very few lay-bys have been provided on the older motor-roads.

Temporary steel overhead roads

It has already been stated that the term "temporary steel roads" is intended to cover both transferable steel overhead roads, and transferable level steel roads.

Dr. Odenhausen has already given a detailed report on steel overhead roads in the September issue of "Acier-Stahl-Steel", so that in the present article we need only mention a few points regarding their construction.

The forerunners of the temporary steel overhead roads were undoubtedly (1) the prefabricated bridges (Bailey, SKR, SE, D, etc.), which, designed for emergencies, are nowadays similarly used at short working sites or in temporary bottlenecks inside and outside built-up areas. These bridges are easy to erect, since their individual components are made of a size that enables them to be handled with very simple equipment. Photographs 2 to 4 show the D bridge scheme and illustrate the possibilities of its use, in this case on road-conversion sites in Berlin, Mungsten and Frankfurt. The weight of a single-storey, single-web bridge designed to "Bridge category 6", amounts to 1.30 t/m.

All the other overhead road solutions have been similarly based on the principle of keeping the weight and cost of the structure within acceptable limits. The overhead roads were thus designed solely for private-car traffic (Fig. 1), *i.e.* "Bridge category 6". The result of this is that:

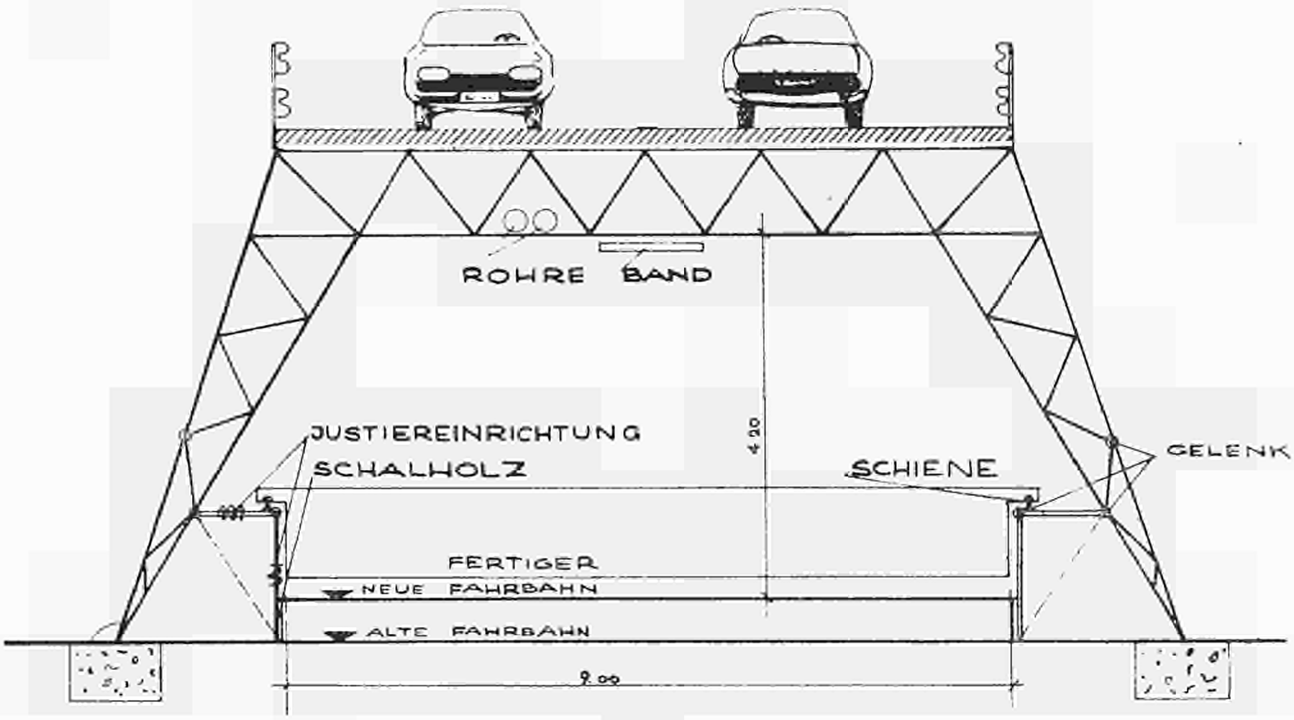


Fig. 5

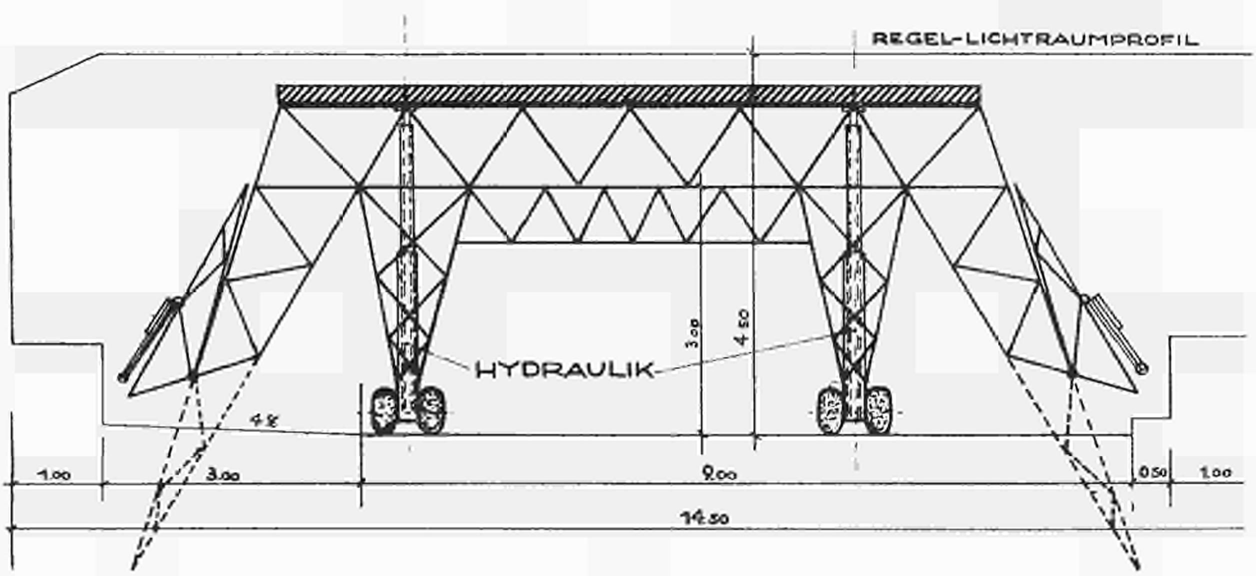


Fig. 6

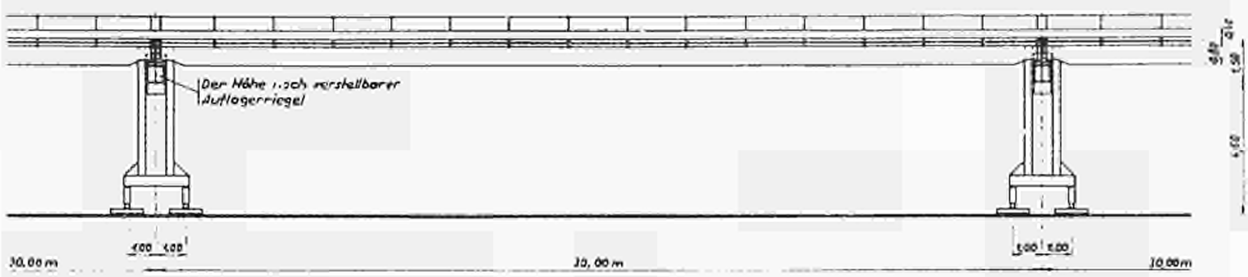
(a) at the entrance to and exit from each working site the traffic has to be made to converge and diverge respectively;

(b) carriageways have to carry traffic in opposite directions.

Both these factors have an unfavourable effect on the flow of traffic — chiefly because of the reduced efficiency of two lanes with opposing traffic as against two uni-directional lanes.

The temporay steel overhead road shown in figures 5 and 6, laid down by the A.-G. für Industrieplanung, consists of tubular steel frame elements of 2.50 m. in length, spanning a road of two traffic lanes. The decking of the overhead road consists of prefabricated roadway slabs, made of steel or concrete and supported by the frames. In the cases shown the overhead road is placed above the new roadbuilding site and as each section of the work is completed, the overhead road weighing about 2.2 t/m; is moved on. This solution is fairly costly; due to this, the separate re-building sections have to be kept very short.

The overhead road proposed by the *Gute-Hoffnungs-Hütte* (Fig. 7)is less complicated. The main structural elements are deck bridges of span 30 m. and width 3 m; constructed symmetrically about the longitudinal axis and the span centres. Individual bridges of smaller span are also provided, so that any desired overall length of bridge can be achieved or any points in the layout reached.



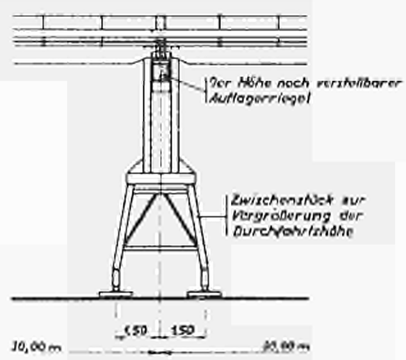


Fig. 7

The overhead road can be built as a frame above the working stretch or as a T-shaped supporting structure on the median strip (Fig. 8). In the case of the frame system, the columns are erected on the concrete edge strips of the working stretch and the frames are of trestle shape, stable in themselves and capable of dealing with horizontal loads in the bridge-direction.

The structure itself consists of a deck plate carried on top of two main girders 1.80 m. apart, with longitudinal and transverse ribs. The construction depth is 0.80 m. Hollow sections have been avoided, in order to keep

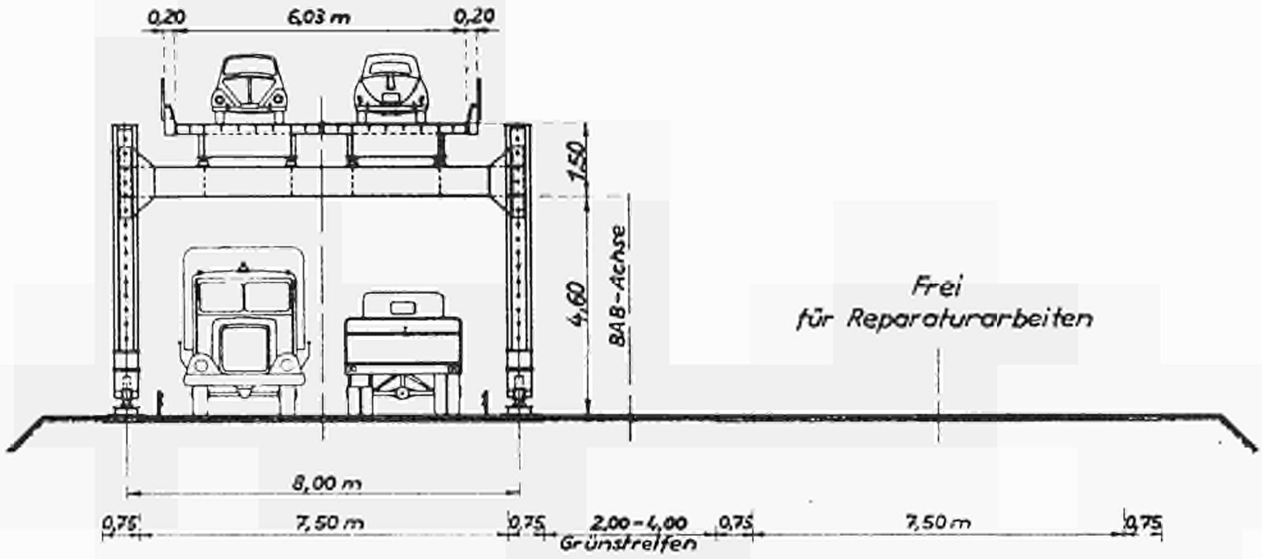


Fig. 8

the structure torsionally flexible and thus enable it to deal with difficulties due to roadway surface joints, both intentional or resulting from inaccuracies in construction. The individual bridges consist of beams on two supports. The height of the roadway of the bridge ranges from 6.10 m. to 18.40 m. and is variable. Fine adjustment can be effected by means of a screwed spindle at the foot of the support (Fig. 9).

The transoms of the frame are designed both as simple supported beams and also as the beams of a two-hinged or rigid frame in order to cover all contingences. A standard frame of 6.10 m. height weighs 9.5 t, while the entire bridge weighs 1.57 t/m.

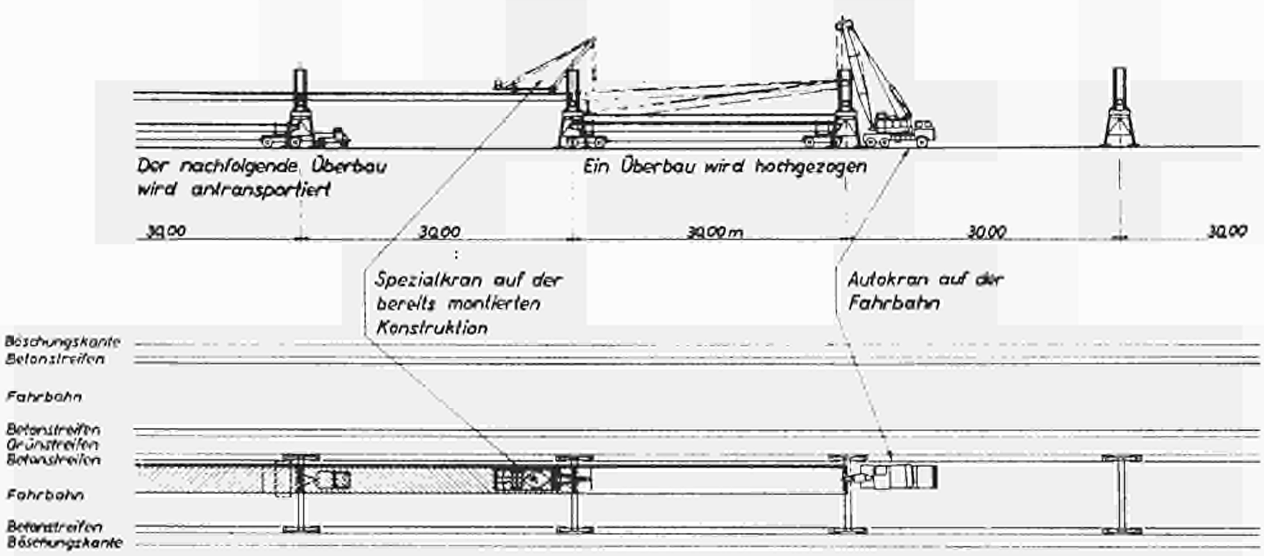


Fig. 9

The firm of *Jucho* also bases the design of its overhead road structure on the principle that only private-car traffic should be transferred to the 2nd level, in view of structure weight (Fig. 10). This firm, moreover, has attached special importance to keeping down the number of different structural elements to a minimum.

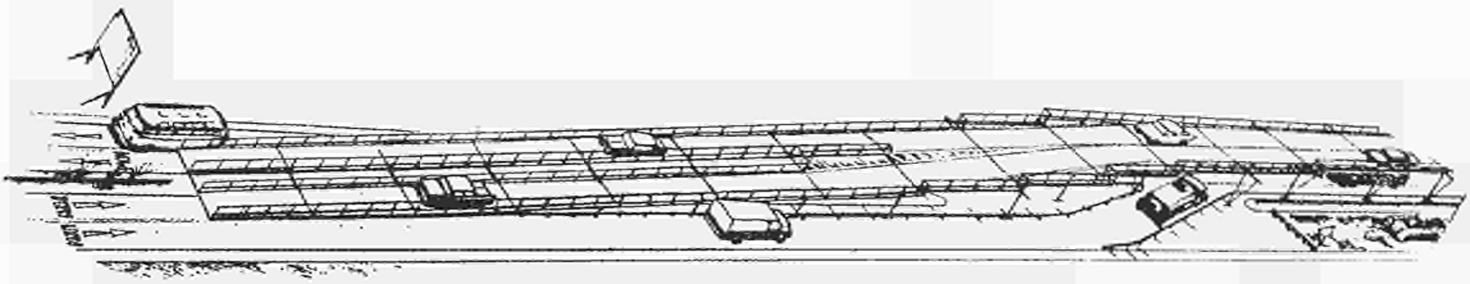


Fig. 10

The bridge deckplate is only 12 m. long and 2.10 m. wide (Fig. 11) and is 0.30 m. in depth without the bearing plate. The bridge span is 11.80 m. and three adjacent plates provide a total bridge-width of 6.30 m., but the safety rails at the side leave about 6.10 m. as useful bridge-width. At the sides of each bridge plate there are two main girders of U-section, 2.10 m. apart overall. The roadway plate is 6 mm. thick and is supported by longitudinal light-weight U-sections of 14 cm. depth. These longitudinal sitffeners rest on transverse beams at 1.47 m. centres and the total weight of steel in the structure is 1.5 t/m.

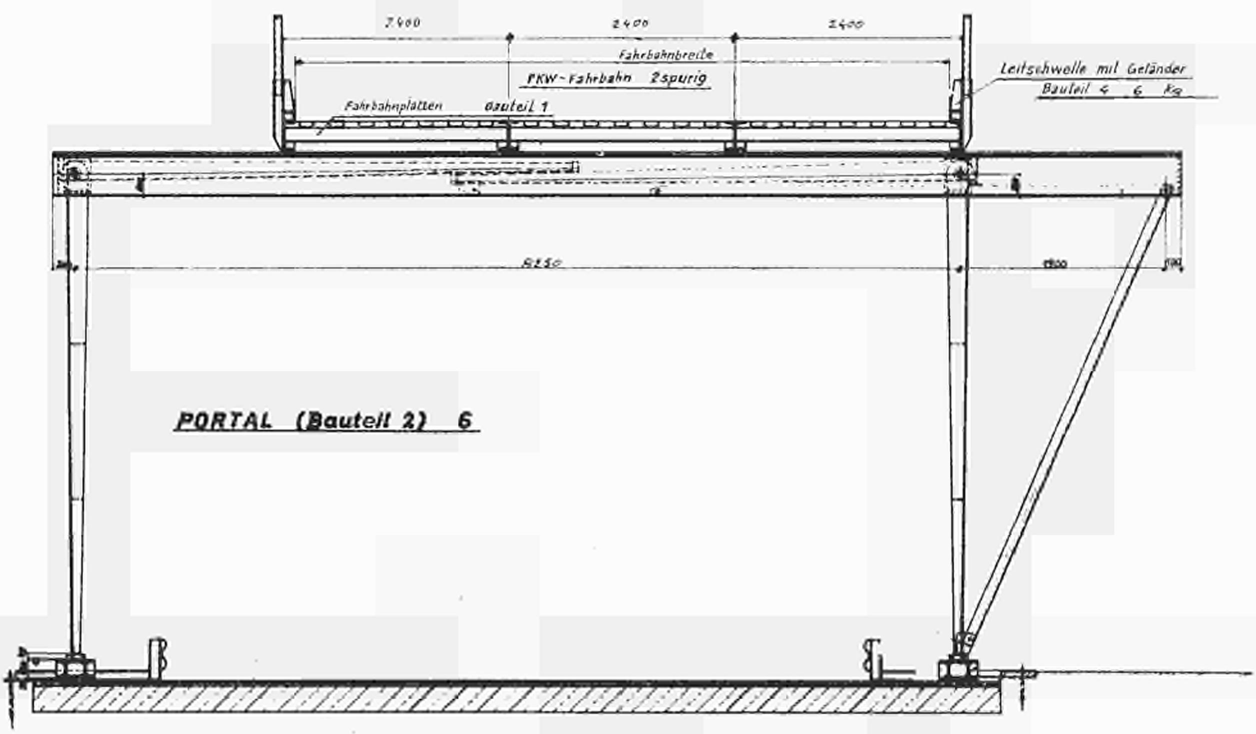


Fig. 11

The portal consists of a crossbeam 10.60 m. long, with two-hinged posts approximately 4.70 m. high, and a system of lateral wind-bracing which takes the horizontal forces acting transversely to the bridge; it is connected by pin joints to the transom (5).

Viewed statically, the portal acts as a single-kneed frame, there being no horizontal forces at the feet of the verticals from the self weight and the traffic load.

The portals are conveyed, folded up, to the working site; the posts and wind-braces unfolding under their own weight when lifted. To stabilize the portal frame in the longitudinal direction of the bridge before the deck has been placed, temporary erection supports are used.

It is possible to erect by means of a mobile crane, and about 480 m. length of bridge can be erected in under 10 hours.

With the exception of the bridge scheme described above, the overhead roads mentioned have not so far been in practical use. This is mainly due to the fact that at reconstruction sites of 4-lane highways we now have availably other methods, not only more economically advantageous but also simpler from the constructional point of view and, above all, offering less traffic risk. It nevertheless remains beyond dispute that the steel overhead road can prove very suitable, where there is only one roadway, or alternatively in built-up areas.

Temporary level steel roads

It can be said that at points of reconstruction of motorways the use of an overhead road is not very appropriate; on the other hand, the use of a level steel road can be very advantageous. In our opinion, the latter should be adopted when :

(a) four-lane traffic is required alongside the construction site and emergency provision of a dual traffic roadway on four lanes is not possible without considerable expense (as is particularly the case when there is no parking lane and considerable depth of road-bed is required);

(b) the reconstruction portion passes over a bridge which has no parking lanes, and

(c) the diverging and converging lanes at the exits and entrances of crossing, intersections and junction points of the Federal motorways are to be lengthened as an emergency measure. The prolongation of the divergence and convergence lanes will in future be necessary in many cases, if traffic is to be kept fluid at such critical points. Figure 12 illustrates this solution but here, however, the steel level road is subdivided into only trhee strips instead of four.

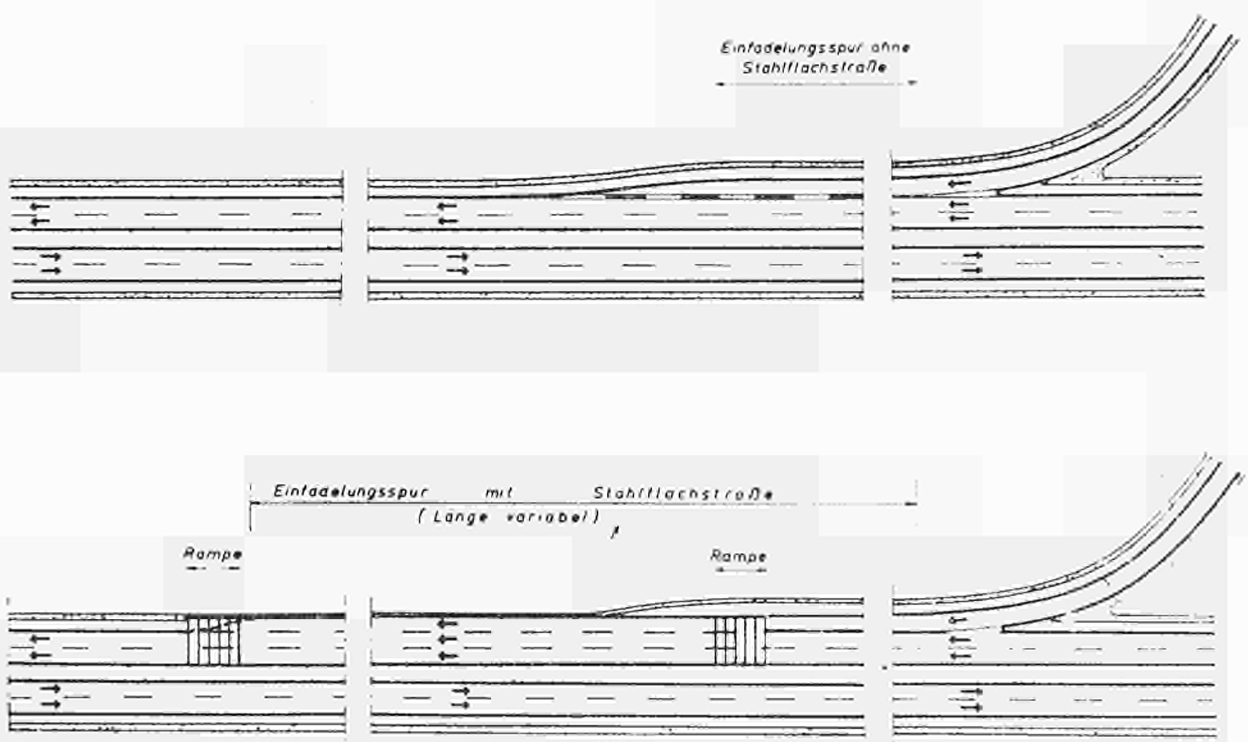


Fig. 12

A further possible application for steel level roads is also to be found in diversions which can be produced by simply laying the plates on fields and meadows.

Technical data for the first temporary level steel roads

The first level steel road constructed in West Germany consists of hollow steel decking units 12.25 m. in length and 2.40 m. in width (Fig. 13), laid close together across the roadway, and connected by means of so-called quick acting couplings and corner bolts (Fig. 14). The deck units allow for two roadways each of 5.75 m. width and each roadway comprises two traffic lanes of 3.25 m. and 2.50 m. width. The boundaries of the roadway are provided by steel kerbs 30 cm. high, fastened by means of hammer-headed bolts.

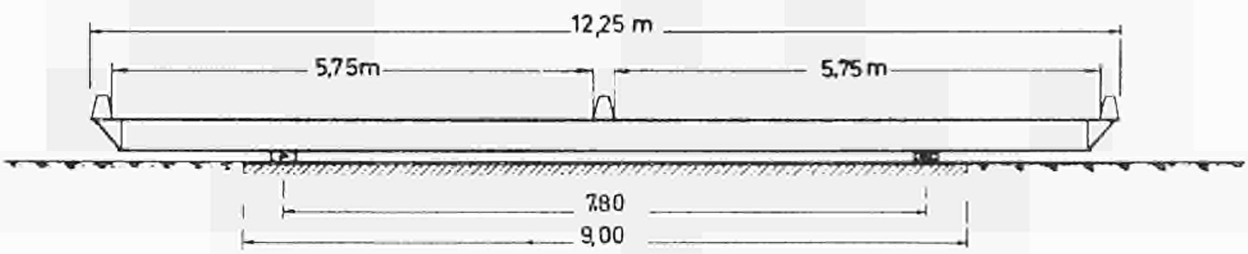


Fig. 13

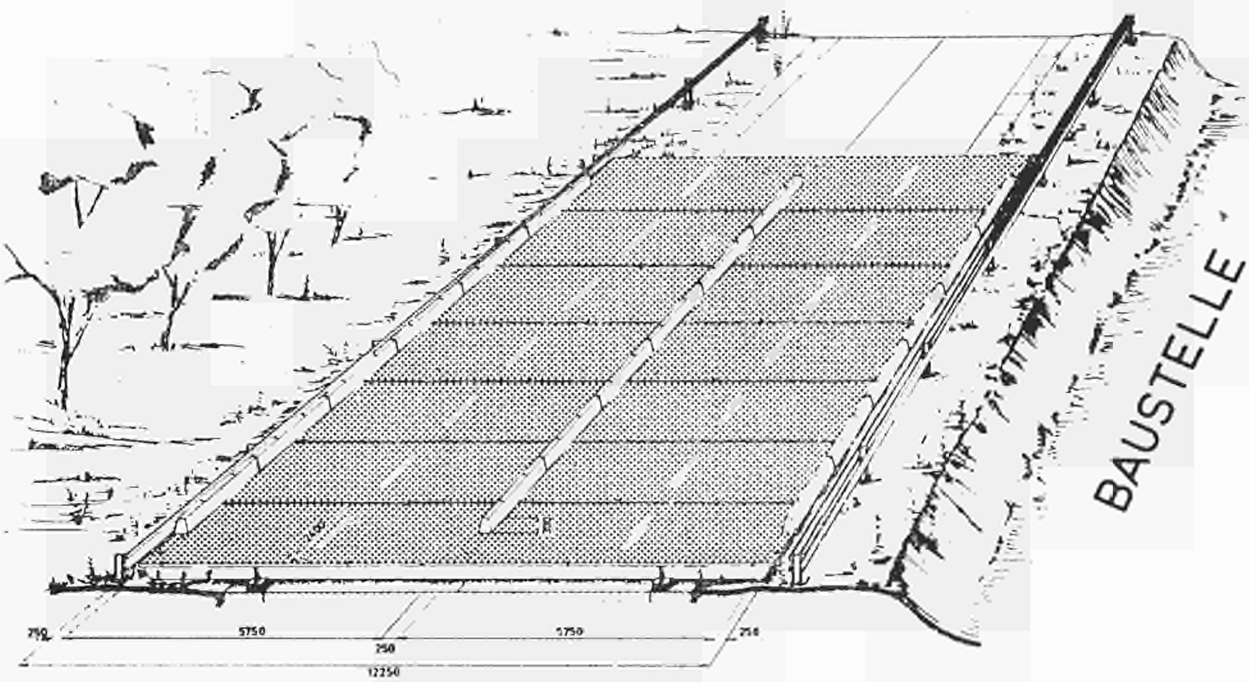


Fig. 14

The dimensions of the plates are of course controlled by the widths required for the traffic strips as well as by the width of the platforms of the standard lorries by which they are to be transported.

The deck plates are supported by two timber sleepers (30 × 12 cm.), normally 7.80 m. apart, to which they are secured by coach bolts. The plates thus project by about 2.20 m. at their ends.

The depth of the hollow units is only 30 cm. (Fig. 15); the ramps which are arranged to give a gradient of of 1 in. 50 are thus correspondingly short.

A 2-mm thick skid-proof coating of plastic is provided on the plates, as a corrosion- and wear-resisting layer.

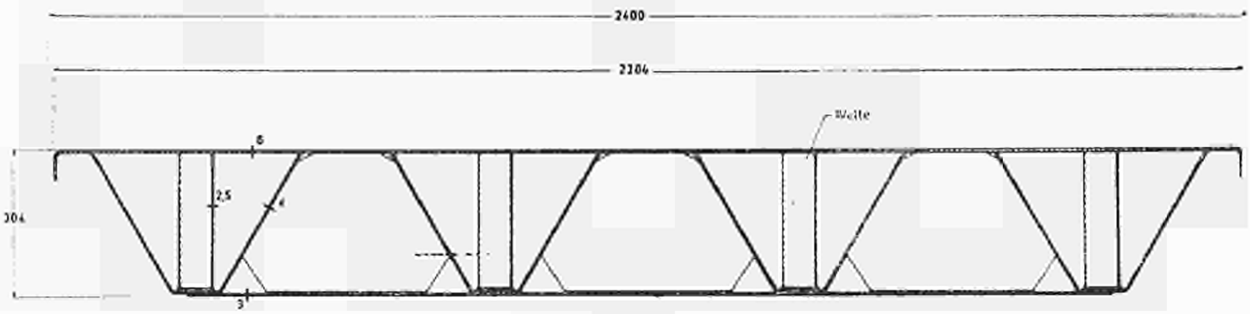


Fig. 15

The dimensions of the decking units also make it possible, in most cases, for the steel road to be tanke through under bridges passing over the motorway (Fig. 16). Where this is not practicable, the traffic can be organized as shown in figure 17; in this case, however, private-car traffic and lorry traffic have to be separated.

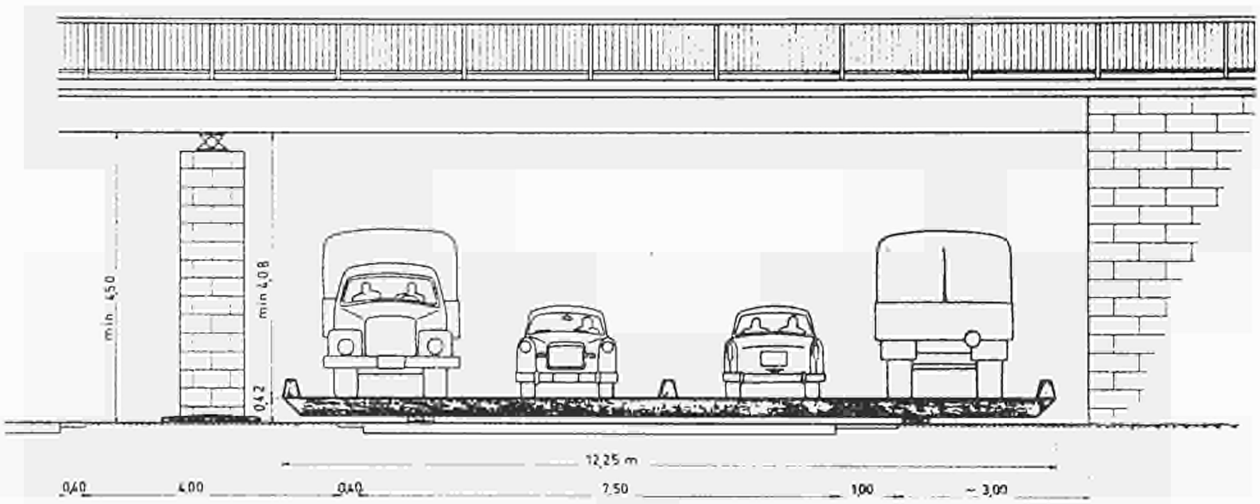
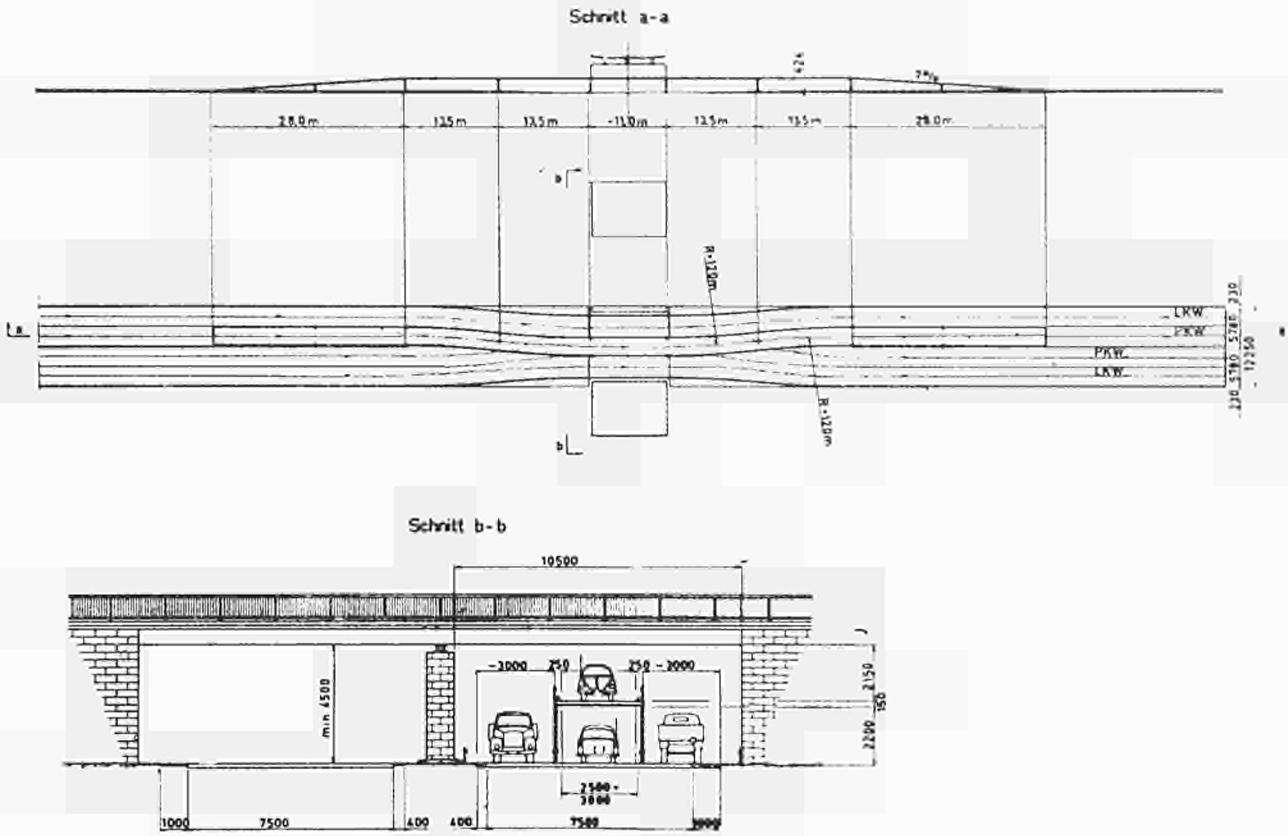


Fig. 16

If the motorway passes over a bridge, the limited weight of the hollow units nearly always makes it possible to lay the steel road on the bridge itself. In this case, however, the steel road must be high enough above the bridge roadway to ensure that the lower edge of the decking is above the existing safety kerbs (Fig. 18).


Schnitt a-a
28,0 m
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PKW
PKW
LKW
12250
Schnitt b-b
10500
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250
250 ~3000
min 4500
2150
150
2200
2500~
3000
1000
7500
600
400
7500
1000

Fig. 17

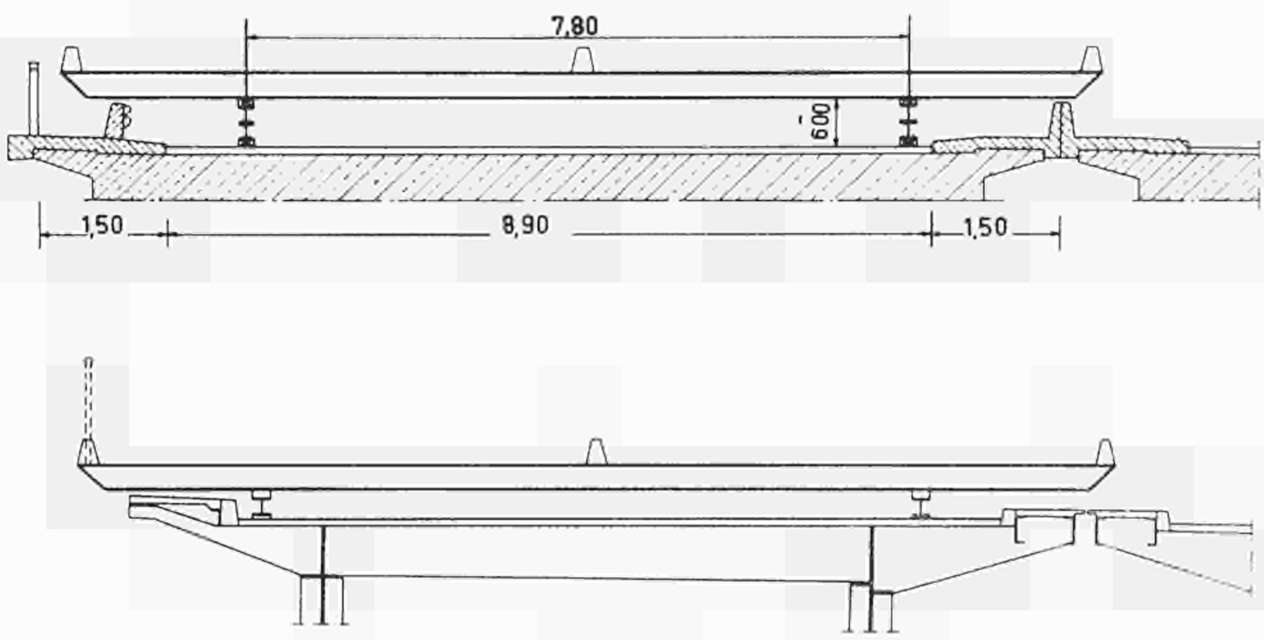

7,80
600
1,50
8,90
1,50

Fig. 18

A continuous straight road is thus provided simply by joining the units. Needless to say, however, the plates can also be laid on bends, up to the relatively small radius of 1000 m. and this is achieved by widening the joints fan-wise between the plates. With a radius of 1000 m. there is a gap of 32 mm. at the outside of the arc, and this is just permissible. If the radius is 2000 m. the gap will be reduced to 22 mm. Smaller radius, on the other hand, can be obtained by the use of trapezoidal units, 2.40 m. wide at one end and 2.20 m. at the other. With these plates, a radius of curvature of 140 m. can be obtained, although this only occurs at entrances and exits to motorways.

The steel level road has been designed to "Bridge category 30" of the German Standard for Design Loads (DIN 1072), so that the steel structure can be used by all vehicles permitted by the traffic regulations.

The steel used for the main structural components is St 52; the upper plate is only 6 mm. thick and the lower plate 3.5 mm. in thickness. Specific fatigue tests were not carried out as they had alreadyb een done with plates built up on similar lines, and the steel had proved quite satisfactory. The durability of the plates, moreover, had long been demonstrated by other practical applications.

The quick acting dowel couplings are of heat-treated steel St52. They are fully capable of transmitting the maximum transverse loads from the traffic, although this is not their purpose.

By the longitudinal sleepers placed under the plates the bearing pressure on the foundation — in this case on the border strips — is evenly distributed. The maximum calculated edge pressure is 7 kg./cm².

To obtain factual information on the actual stresses and strains which would accur in the plates, tests ware carried out on the first steel road laid, and these showed that, with the usual static live load and in test runs with a 3-axle test vehicle, having a front axle weight of 4.5 t., and 8.6 t. on each of the two rear axles, the stresses in the top and bottom plates were always below 3 kg./mm².

Erection of temporary steel level road

The laying of the steel road is simplicity itself, as special care was taken at the design stage to ensure that traffic would not be unnecessarely impeded. The laying or transference of the units is therefore no longer a technical problem but merely one of organization.

It is advisable to keep the structural elements in readinness in advance on a storage site near the place where they are eventually to be used. From this storage site the plates may be conveyed to the working site on lorries which can each carry five plates.

The placing of the units starts at the centre of the section being rebuilt, and two teams work outwards from this point, in opposite directions, towards the ends of the section. Each team should preferably be provided with a mobile crane of 60 tm. lifting capacity.

The structural elements are conveyed to the site from one direction (Fig. 19) and in each case two loaded vehicles arrive simultaneously. The first proceeds, over improvised ramps of light metal or wood, over the plates already laid, to the other end of the site. The ramps are then moved to one side and the units laid. As soon as the vehicles have been unloaded, the temporary ramps are again placed in position; the empty vehicles depart, and the first of the second two loaded vehicles which have just arrived proceeds at the same time over the units already laid to the other end. This process is repeated until the entire road has been laid. The final ramps which, with the exception of the junction units, have a gradient of 1 in 50, are then placed at the two ends. The 4 lane road can now be made available to traffic and the adjacent roadway repaired. After the termination of the work, the steel road is dismantled in the reverse order. If the length of the section of road being rebuilt exceeds that of the steel road, the plates have to be transferred at the appropriate stage. There are two possible ways of doing this, according to whether the road-building operations are carried out in separate sections (in which case the length of each separate section must as far as possible correspond to that of the steel road) or on the continuous system. In the former case the units, after the completion of a section of the motorway, are taken up from the ends and, as already described, transferred to the new working site (Fig. 20). The four-lane traffic over the steel level road is stopped during this transfer and continues in twolanes in opposite directions over the new roadway of the completed sections. The foundation plant can be placed on storage sites, at the side during this period.

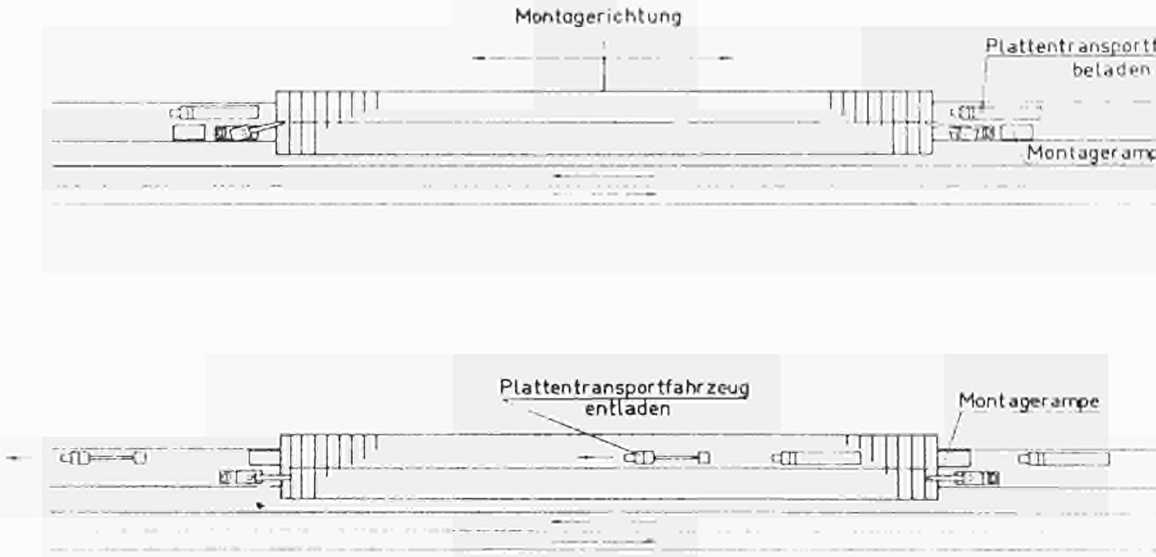


Fig. 19

If, on the other hand, continuous roadbuilding is required for economic or practical reasons, then as soon as a section of the motorway has been completed (and this must naturally be shorter than the steel roadway), the traffic is taken in two lanes over this section and also over that part of the steel level road which is to remain in place. In this case lateral ramps are used to bridge the difference in level between the motorway and the steel road (Fig. 21). After the change of use, the roadway grillage is dismantled on one side and the dismantled units are re-laid on the other side.

Traffic in connection with the rebuilding operations also runs in two lanes over that part of the steel road which remains in place.

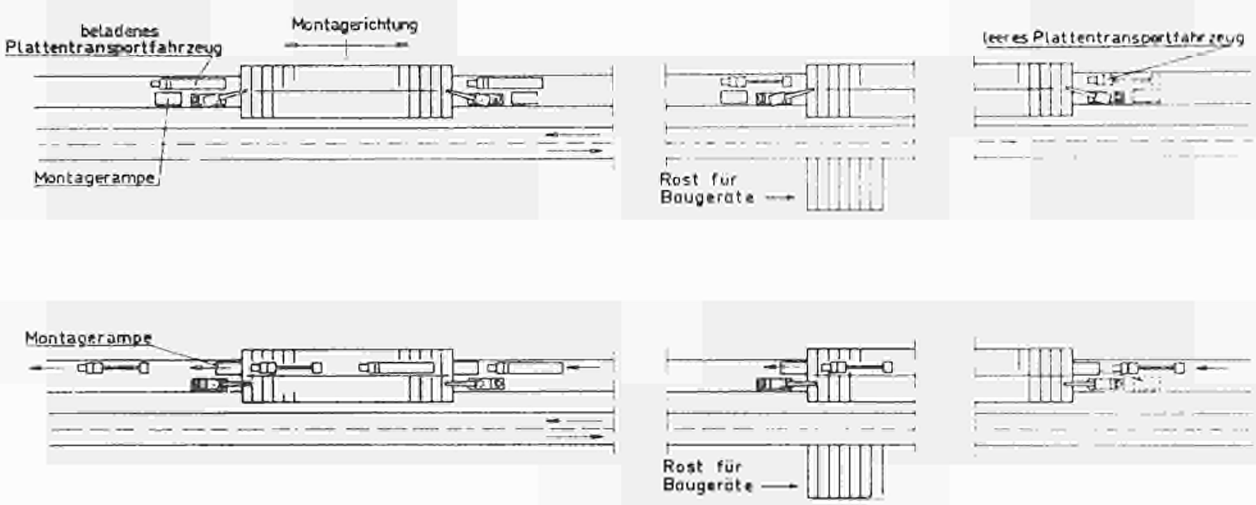


Fig. 20

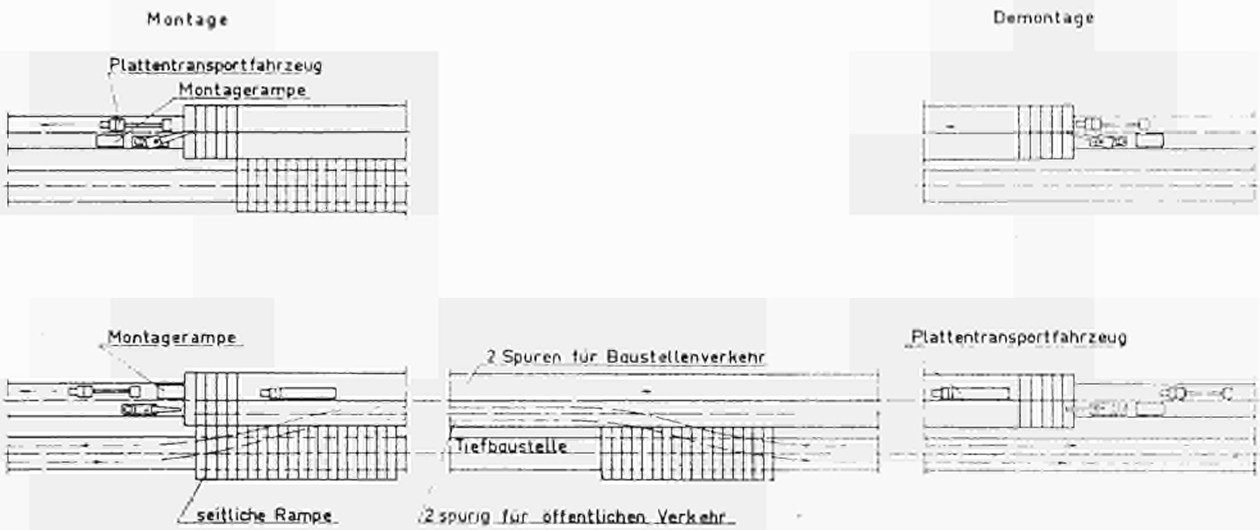


Fig. 21

First application of a temporary steel road

In the spring of 1964 we used a flat steel road for the first time, on the Frankfurt-Kassel motorway in the vicinity of Butzbach, and since then it has been re-used on two more occasions. The approximate length of this road is 1000 m. It was first transported to and stacked at a storage site 9 km. from the place where it was to be used.

A week-end had been set aside for its installation, since in West Germany heavy lorries do not run at the week-ends. The road, with its ramps, was to be laid in 24 hours, which meant that any uneven places on the lateral edge strips, on which the plates were to rest, would first require to be levelled.

Krupps used 11 semi-trailers for the transport of the plates, 1 gantry crane at the storage site, 2 mobile cranes at the working site and a total personnel of 130, which, for this first operation, laid the entire road in 17 hours.

Each trailer arrived at the working site with five plates and was unloaded by well-trained personnel. As soon as a plate had been laid it was connected to the adjacent plate by means of four double-headed dowel couplings (6, 7, 8, 9 and 10). This method of coupling was extremely easy and contributed considerably to rapid installation. Radio communication between the two teams, also between them and the storage site, ensured smooth transport operations for conveying the units to the site as well as accurate co-ordination of the work.

As soon as the units had been coupled together, the side and central kerbs were fixed, the ramps laid and the steel road then made available to traffic. It was then possible to start the renewal operations on the other roadway, and the flow of traffic remained undisturbed.

The subsequent transference of the flat steel road proceeded equally smoothly. Here, however, the route had to pass under bridges of which the clearance height could not be reduced. The steelwork was therefore interrupted in the zone corresponding to the bridge and taken down to the roadway of the autobahn by means of ramps (11). This, however, meant that the soft ground below the bridges had first to be made firm over a width of approx. 12 m.

Results obtained with the first temporary steel level road

Both the method of assembly and the application of the steel road have proved reasonably satisfactory, apart from minor structural defects to which any new structure is subject and which are only detected in practical use. Despite peak loadings of up to 40,000 vehicles a day, or 2,500 an hour, no traffic-jams occurred. Neither have there been any accidents due to the steel level road.

The rattling and reverberation of the plates was however, a disadvantage. The causes of this have been determined and can to a large extent be overcome. For example, the timber sleepers, only 2.40 m. long in each case, did not always bear firmly on the side strips and these, at any rate on the older roads, are not very strong and in most cases rest on a sandy substratum. This, resulting in an excessive surface pressure and, finally, in partial destruction of the side strips, contributed considerably to the rattling of the structure.

The steel units themselves have proved too stiff in torsion, and less rigid units are certainly more satisfactory. Units are now being designed which offer the considerable advantage of being lighter in weight and therefore less costly.

The central kerb is somewhat too high (30 cm.) and a height of 15-20 cm. should be sufficient, in view of the fact that the door of a motor vehicle should not strike it on being opened.

No conclusive statement can as yet be made on the surfacing of the roadway. Fifteen different plastic coatings were applied to the first steel road of this type in order to determine their behaviour under traffic conditions. The two coatings with a tar base have not proved satisfactory; the other coatings, on an epoxy-resin base, have so far shown no defects.

Further development of the temporary steel level road

The experience gained with the first steel road has naturally been evaluated with a view fo finding the optimum structural solution. In the future, for example, the lower plate will be dispensed with, which means

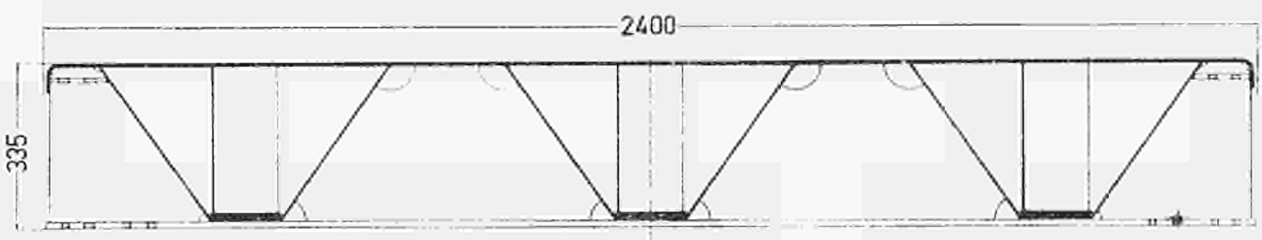


Fig. 22

that an orthotropic stiffened plate, of which the horizontal plate is 7 mm. in thickness, will be used as the structural element (Fig. 22). The booming of the units will thus be practically eliminated. No more dowel couplings will be used between plates as the plates will be simply bolted to the two bearing girders. The supporting system itself, on the other hand, will be on the same principle, but the timber sleepers will be replaced by continuous steel girders of which the upper flanges will be rubber coated. This will largely eliminate the rattling of the deck and ensure a better distribution of the load.

A suggested increase in width of units to about 3 m. will not be put into effect owing to limitations imposep by transport. There are other improvements of secondary importance, which it is unnecessary to enumerate or explain.

Temporary steel level roads required during the coming years

The level steel road has been in use in West Germany since April 1964. It would be unwise, after such short experience, to give a final verdict, or to draw any final conclusions as to the future prospects. It may be assumed, however, that as this road has passed its test both technically and from the traffic point of view; the cost involved will be the deciding factor for the future. If we are successful in using it very frequently without loss or damage, it will certainly gain a firm footing on our road-repair sites.

The next few years, moreover will show whether the steel road cannot and should not perform other functions, such as those arising on urban thoroughfares.

Concluding remarks

The advantages and drawbacks of temporary steel roads have been indicated in considerable detail If we have drawn particular attention to their weak points, our object has not been to impede by negative criticism a line of progress and development which has just commenced, on the contrary, the purpose of this paper has been to show where the main efforts must be concentrated in order to provide the widest possible field of application for the temporary steel road of the future.

Description of photographs

1 — Components of D bridge.

2 — D bridge open to traffic (Berlin).

3 — D bridge open to 2-lane traffic near Müngsten

4 — Model of D bridge at Platz der Republik Frankfurt.

5 — Model of temporary overhead steel road at Jucho.

6 — Transportation of deck plates to building site near Bützbach. (Frankfurt-Kassel motorway

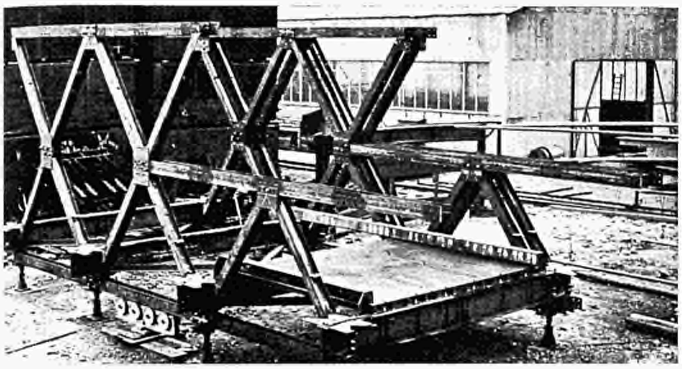
7 — Transportation for re-use of temporary steel road in the vicinity of Bützbach.

8 — Transportation of hollow steel decking units.

9 — Assembly of hollow steel decking units.

10 — Assembly of hollow steel deck units connected by quick-acting couplings.

11 — Section of steel road passing under bridge without interfering with headroom.



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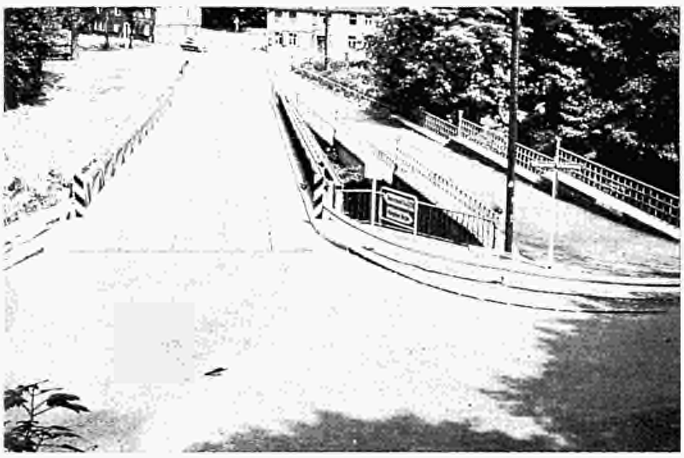

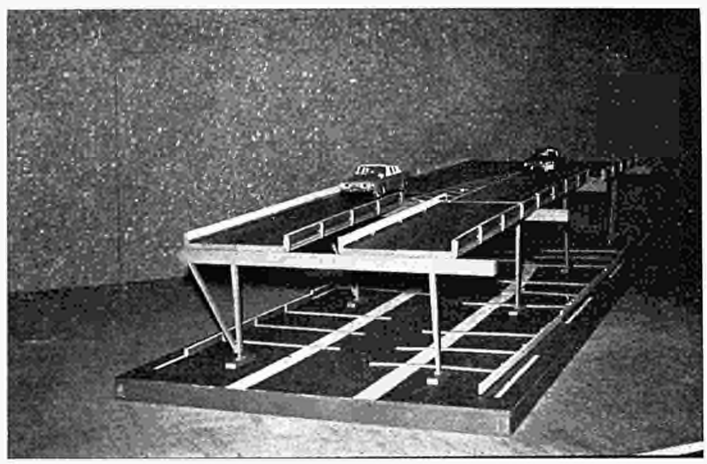
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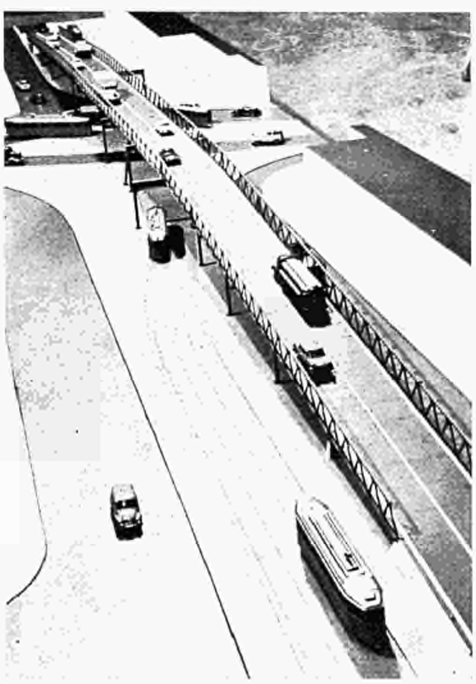
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Helmuth ODENHAUSEN

Modern Footbridges in Steel Construction

(Original text: German)

The proportion of pedestrians killed in road accidents — measured in relation to the total number of road casualties — is constantly and alarmingly increasing. Figure 1 illustrates this frightening state of affairs in graph-form for the German Federal Republic. The diagram indicates the relative proportion of pedestrians among the fatal casualties on the roads. In 1957 this proportion was 24.4%, in 1960 it was 31.1%, and in 1962 it had risen to as much as 34.5%. In these figures there is more, particularly, a steady increase in the number of older people killed. The greater danger to the elderly is due to the higher traffic density, to the longer expectation of life, and to the increasing lack of road-sense and care on the part of the road users, which is caused chiefly by the adverse effects of modern civilisation.

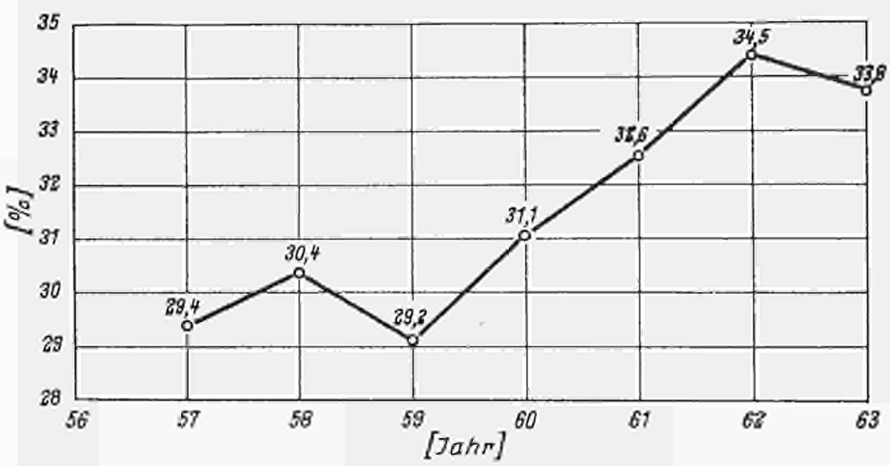
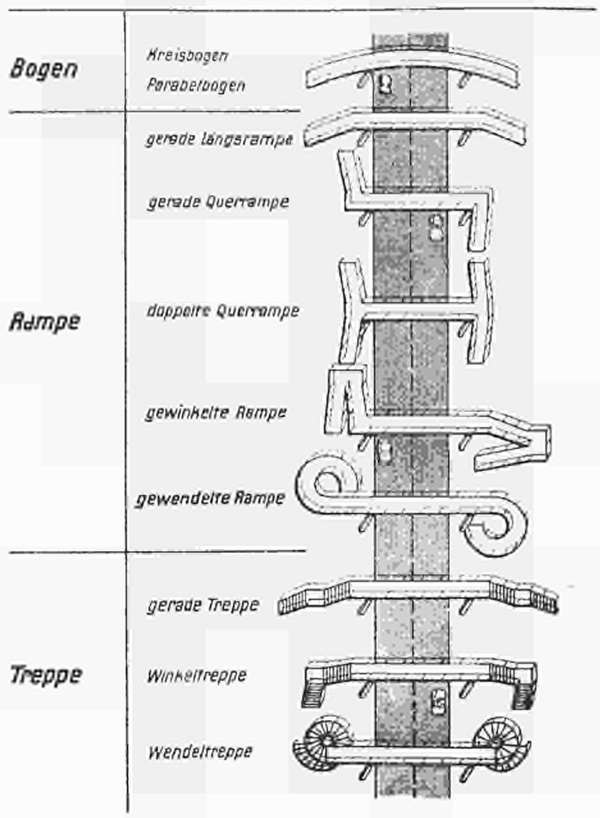


Fig. 1

By far the great majority of accidents in which pedestrians are involved occur on crossing the road. Zebra crossings and street-level pedestrian crossings controlled by light signals do, indeed, relieve the situation to some extent for all road users, but they cannot with certainty obviate the danger to pedestrians crossing the highway. There still remains, the risk of human fallibility on the part of the already harassed motorists. In addition, such street-level pedestrian crossings interrupt the flow of traffic and thus constitute an added hindrance.

In the case of railways and waterways attempts have long been made to eliminate level crossings. For example, the last pontoon bridges over the Rhine have disappeared, and ferries are more and more being replaced by fixed bridges. In recent years the German Federal Railways have, from their own resources, spent over 50 million DM per year on the elimination of level crossings. The application of the same principle to the crossing of highways by pedestrians, namely, by routing them at an elevated level across the road, leads to the provision of footbridges.

Footbridges have indeed proved to be a reliable and safe remedy, especially in dense and fast road traffic.



arch	segmental arch
	parabolic arch
ramps	sloping ramps, in line with the bridge
	sloping ramps, at right angles to the bridge
	double sloping ramps, at right angles to the bridge
	sloping ramps with landings
	spiral ramps
stairways	straight stairways
	corner stairways
	spiral stairways

Fig. 2

A feature that nearly all footbridges for conducting pedestrians over road traffic have in common is that the users have access to them at street level and also step off them at street level. Hence the footbridge normally comprises three parts : an upward-sloping part, a horizontal part, and a downward-sloping part. For the sloping parts a gradient of 15% is regarded as the acceptable upper limit. Figure 2 shows schematically the structural possibilities for overcoming the differences in level that commonly occur with footbridges. The ascending and descending parts, *i.e.*, the sloped portions of the structure, may be arches, ramps, stairs or escalators. These may be in line with the centre-line of the bridge, or be set at an angle thereto or, in addition, be curved. Finally, they may have a constant or, alternatively, a constantly varying slope. The latter case is, for example, encountered in footbridges in the form of a flat arch. In such bridges there is no horizontal central part. Figure 3a gives a comparison between a flat parabolic arch bridge and a corresponaing girder bridge with straight ramps. Generally speaking, the arched footbridge is architecturally satisfactory because of its geometric shape, as will also be evident from subsequent illustrations. However, it has the apparent disadvantage that the crown of the bridge has to be at a height somewaht greater than the minimum clearance. This disadvantage is, however, offset by the favourable variation in slope, from the point of view of the human physical effort required, of the flat parabolic arch bridge. As can be seen in the lower part of this illustration, (Fig. 3b) the straight ramp of the girder bridge has a constant gradient, whereas the gradient of the parabolic arch is, it is true, initially greater but then steadily diminishes to zero. This compensates for the physical fatigue caused by the brief effort of climbing.

A disadvantage of the flat arch bridge, as contrasted with the girder bridge with straight ramps, however, is the considerable amount of space required on both sides of the roadway. This indicates the fact that the choice between the flat arch bridge and the straight girder bridge is fundamentally governed by the amount of space available on each side.

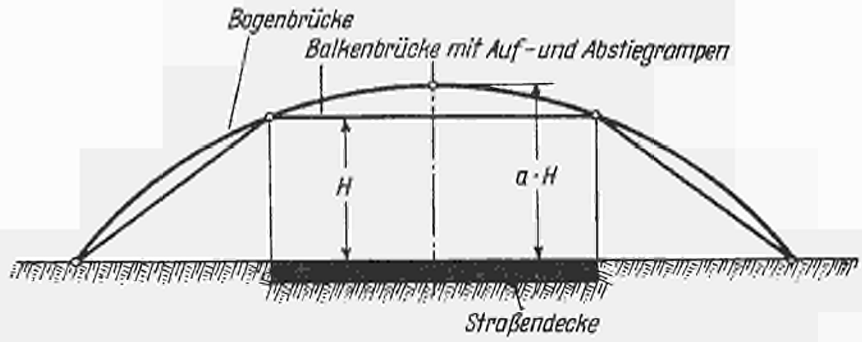


Fig. 3a — Elevated pedestrian crossing of the arch bridge or girder bridge type

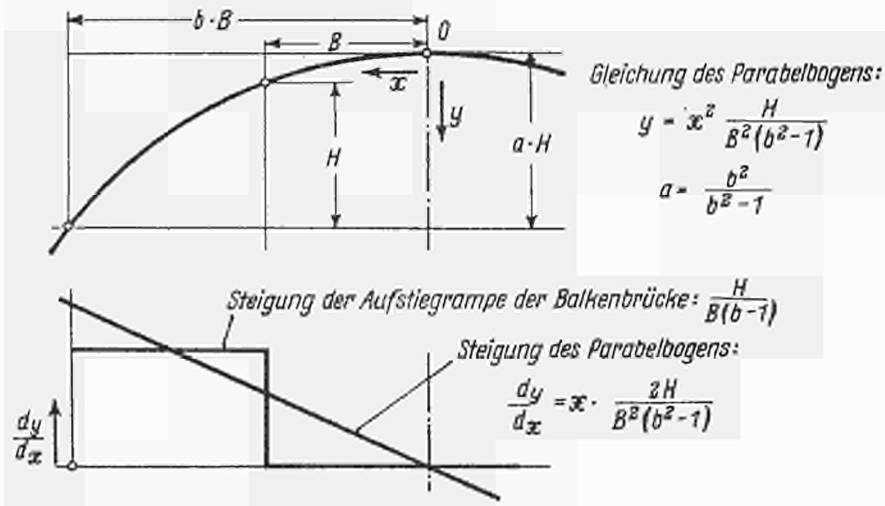


Fig. 3b — Comparative gradients of the parabolic arch bridge and of the approach ramp of the girder bridge.

The bridge systems usually employed for modern footbridges are schematically summarised in figure 4. I shall presently supplement this summary with a large number of examples of bridges actually constructed (1-62).

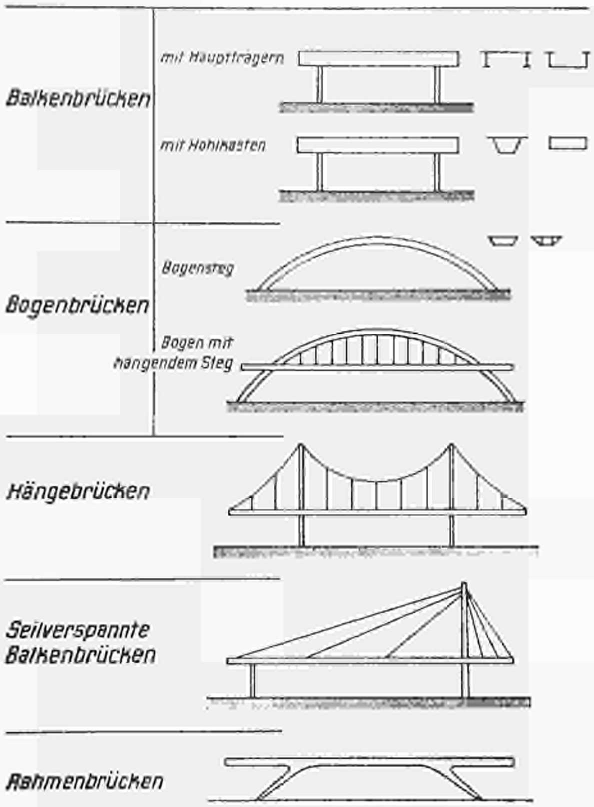
As regards the supports, the most commonly employed forms are shown in figure 5.

For footbridges, steel construction offers some substantial advantages : the bridges are prefabricated in the works and are quickly erected on the site. Interference with road traffic can be almost entirely avoided by conveying the large prefabricated units and erecting them during the night hours when there is little traffic. Another advantage is that light steel bridges can easily be dismantled and re-used elsewhere. This fact is particularly important with regard to road widening or other road development schemes. For example, when the Cologne — Leverkusen motorway was widened from four to six traffic lanes, the removal of concrete flyovers was very costly indeed.

With regard to the steel footbridges hitherto constructed, the use of plate girders, of rolled steel sections, and of tubular construction is to be distinguished.

Architectural treatment and the need for the appearance of the bridge to harmonize with the urban or rural scenery are of the utmost importance, particularly in connection with footbridges for pedestrians over roads, since these bridges differ from ordinary bridges in that they do not cross a waterway but level ground.

The following illustrations will serve primarily to give you an idea of the numerous possibilities that steel construction has to offer from the architectural, traffic engineering, and structural points of view.



Girder bridges with	main girders
	box-girders
Arch bridges	arched deck
	arches with suspended deck

Suspension footbridges
Cable-stayed girder bridges supported by wire ropes from a pylon
Portal frame bridge

Fig. 4

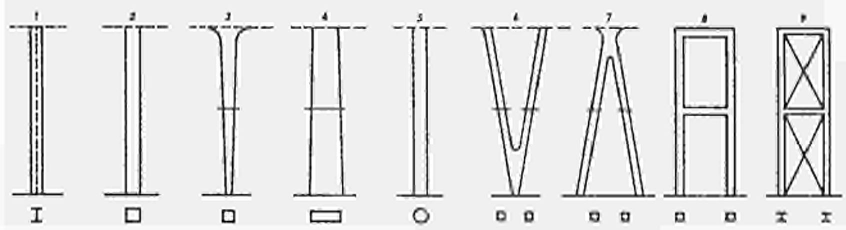
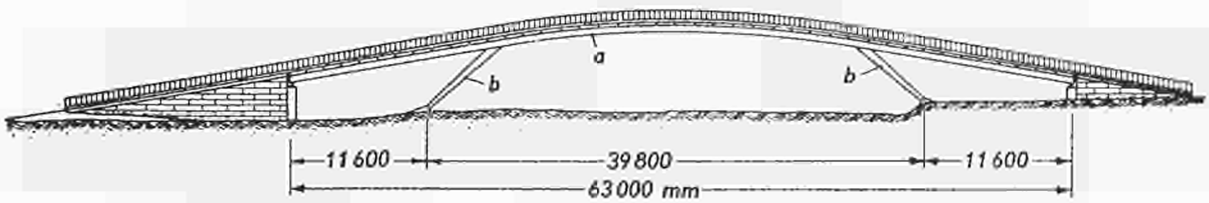
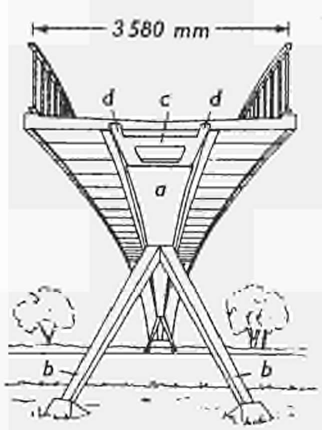


Fig. 5





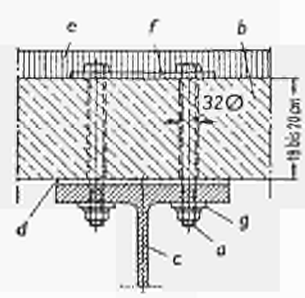


Fig. 6 — Box-section central girders; A-type supports.

It trust that, with the help of these mainly visual aids, I have succeeded in giving you an idea of the numerous possibilities of footbridges in steel construction.

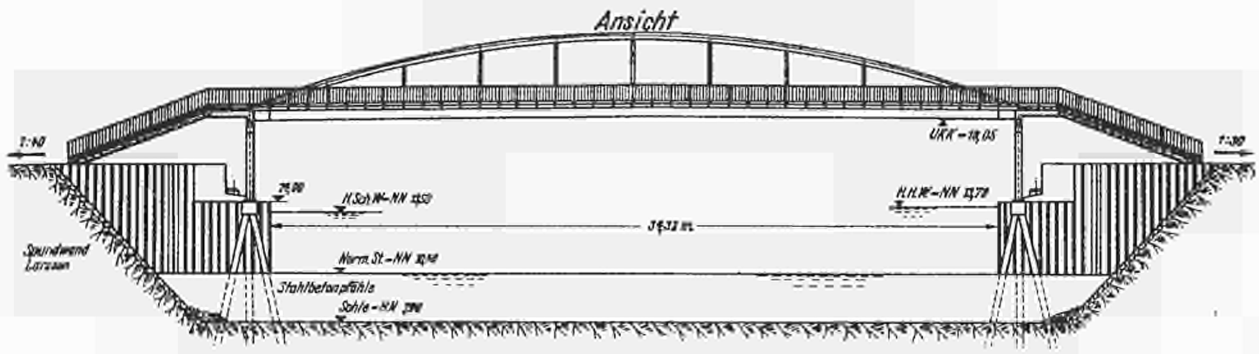


Fig. 7 — Middle support arch bridge over a river channel near Meppen. The deck forms the tie-member of the arch girder. As can be seen, the arch was prefabricated as a single whole.

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Description of photographs

1 — Footbridge at Stuttgart.

2 — Girder bridge; splayed portal frame; the two main girders, of tubular construction, also serve as parapets; straight access stairs with landings.

3 — Footbridge over motorway near Gütersloh.

4 — Portal frames with corner bracings; here too the lateral lattice girders serve as parapets; the access stairs, which are provided with landings, are set at an angle.

5 - 6 — Designs for footbridges with access stairs set at right angles, for installing at points where the footpaths are narrow.

7 — Footbridge over railway tracks at Mannheim. Composite construction; box-section central girders; A-type supports.

8 - 9 — Footbridge over road and tramlines at Dortmund. Box-section central girder, V-type supports.

10 — Footbridge at Alhambra, Cal., U.S.A. Helical ramps on both sides. For reasons of economy the bridge was not constructed as an arch bridge, though that would have been architecturally preferable. The curved arch has been replaced by three straight secant sections.

11 — Stairs with landings as intermediate ascents on a footbridge at Brussels. The structure comprises two parts which are connected by means of hinges both at the foundations and at bridge deck level.

12 - 14 — Footbridge, comprising two parts, over the Messe-Schnellweg, an express road at Hanover. Box-section frame structure for the main span, box girders for the subsidiary span. Additional descent between the two parts of the bridge.

15 - 17 — Footbridge at Orly Airport, Paris. Box-section structure : two-pinned arch as structural system carrying straight deck components.

18 - 19 — Footbridge in the Westfalenpark (Wesphalia Park), Dortmund. Arched box-section frame and deck in the shape of a flat arch.

20 - 21 — Flat arch bridge over the Ruhrschnellweg (Ruhr Express Road) at Dortmund. Central box girder, A-framed supports.

22 - 24 — Flat arch bridge over the Ruhrschnellweg (Ruhr Express Road) at Wattenscheid. Non-dowelled composite construction, central box girder, concrete slabs secured with high-strength bolts, A-framed supports.

25 - 26 — Flat arch bridge over the Ruhrschnellweg (Ruhr Express Road) at Dortmund. Central box girder, three downward-tapering intermediate supports, and an additional rigid support brace.

27 - 29 — Flat arch bridge over the Ruhrschnellweg (Ruhr Express Road) at Essen-Frillendorf. The bridge crosses four road traffic lanes and two tramway tracks. Pre-stressed concrete helical ramps, arched deck of non-dowelled composite construction comprising a steel box girder and precast reinforced concrete slabs; three steel tubular supports corresponding to 2 × 27 m. span. Erection without interference with traffic.

30 — Flat arch bridge over an express road at Düsseldorf. Central box girder; three box-section supports.

31 - 33 — Flat arch bridge over the Hanover-Hamburg motorway. Main girders curved in three dimensions. The ramps are set at angles in opposite directions. Favourable solution for a footpath intersecting the motorway at an acute angle.

34 — Arch bridge with suspended deck over an express road at Cleveland, Ohio, U.S.A.

35 - 36 — Middle support arch bridge over a river channel near Meppen. The deck forms the tie-member of the arch girder. As can be seen, the arch was prefabricated as a single whole.

37 - 39 — Footway originally erected at the German pavilion at the World Exhibition at Brussels in 1958, now crossing the motorway near Duisburg, Torsionally rigid box-section lateral girder designed as a cable-stayed beam, with a short return arm (anchored at the end) and a long cantilever arm; needle-shaped tower, 50 m. high, with streamline-shaped box-section.

40 - 41 — Cable-stayed girder bridge at Stuttgart. The deck is formed by a wide box girder only 565 mm. in depth; the tower is 23 m. high; the deck is bifurcated in front of the tower, at the ascent on the one side.

42 - 44 — Cable-stayed girder bridge over the Glacis-Chaussee at Hamburg. Box girder with trapezoidal cross-section of variable depth; tower of triangular cross-section, 28.6 m. height, on centre-line of bridge, main span 28.6 m.; reinforced concrete stairs.

45 - 47 — Footbridge in the grounds of Cornell University, Ithaca, N.Y., U.S.A. Portal-type towers at both ends; span 280 m.; lateral lattice stiffening girders also serve as parapets.

48 — Bascule footbridge near Leyden in Holland. This is an example of a movable bridge.

49 - 50 — Vertical-lift bridge for pedestrians between Manhattan and Ward's Island, N.Y., U.S.A. Another example of a movable footbridge. Overall length 292 m.; length of vertical-lift centre span 100 m.; machinery installed at tops of portals.

51 - 52 — Footbridge over the Glacis-Chaussee at Hamburg. An example of a structure assembled from prefabricated and standardised components.

53 - 54 — Dismantable belted footbridge over an express road at Hanover.

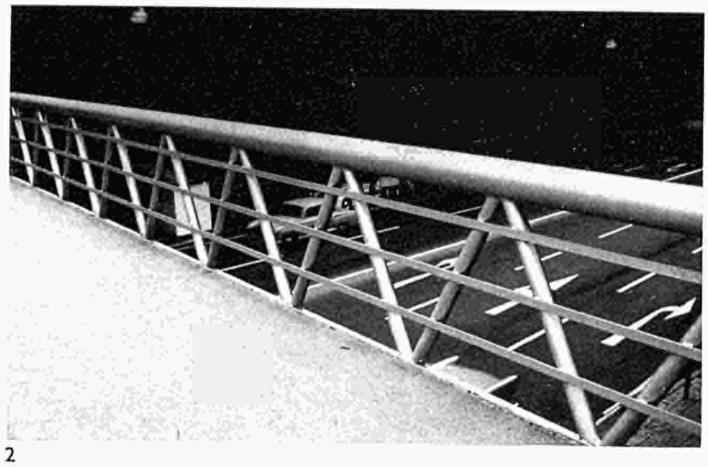
55 - 56 — Another example of a structure assembled from prefabricated and standardised components. Supports are "space" structures of I.R.E. rolled steel sections and tubes; lateral main girders and wind bracings also of tubular construction.

57 - 58 — Demountable, bolted footbridge at Frankfurt. Erected during constructional operations for the underground railway.

59 — Temporary footbridge assembled from steel tubular scaffolding components, at Leverkusen. The two lateral main girders are here located above deck level.

60 — Temporary footbridge assembled from steel tubular scaffolding components, at Hanover. The centrally located main girder is under the deck.

61 - 62 — Temporary arch bridge assembled from falsework components, erected over an express road at Hanover.



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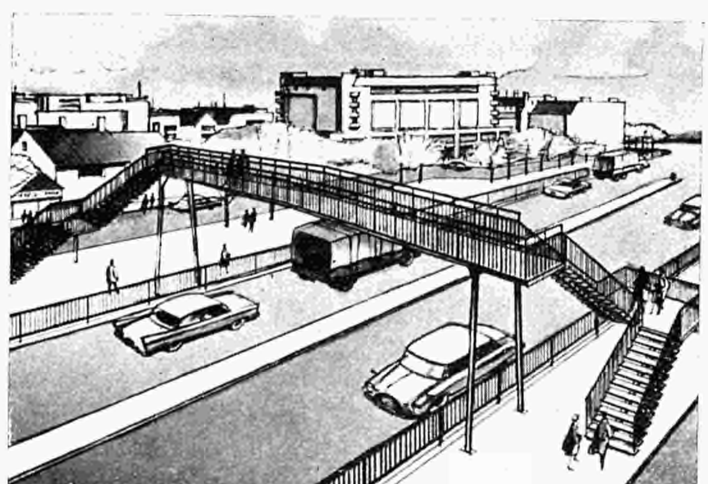

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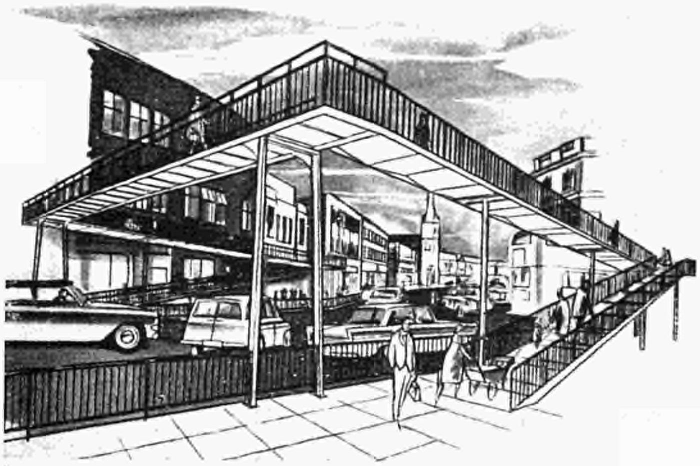
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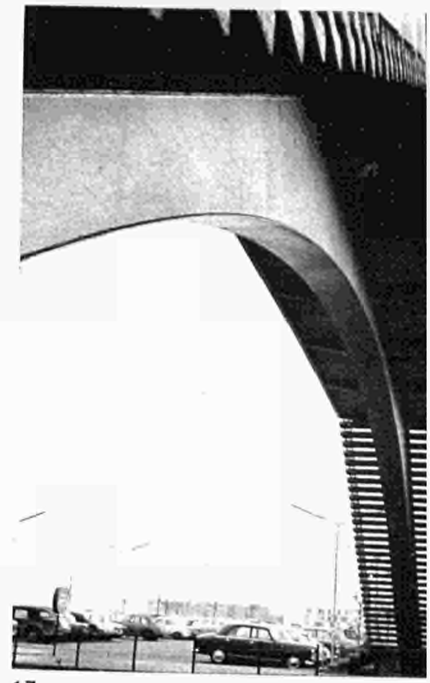
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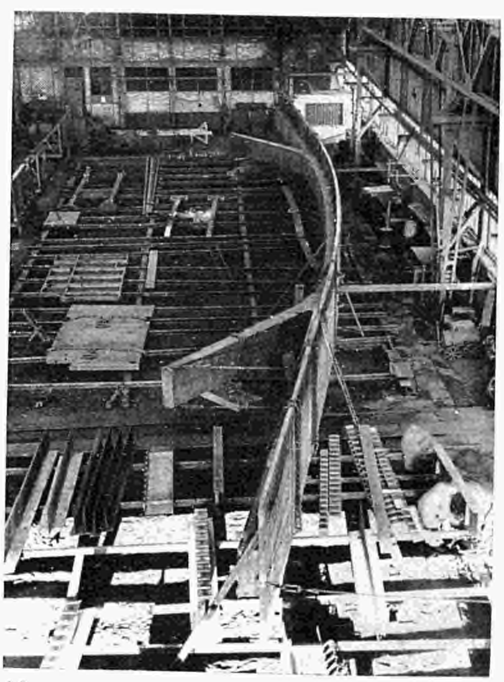
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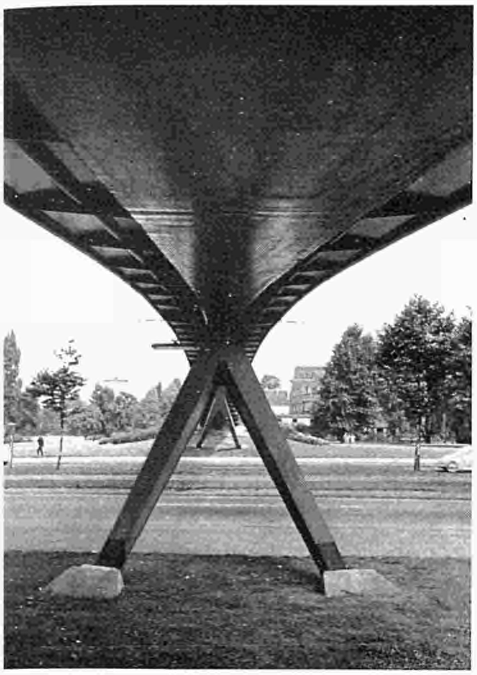

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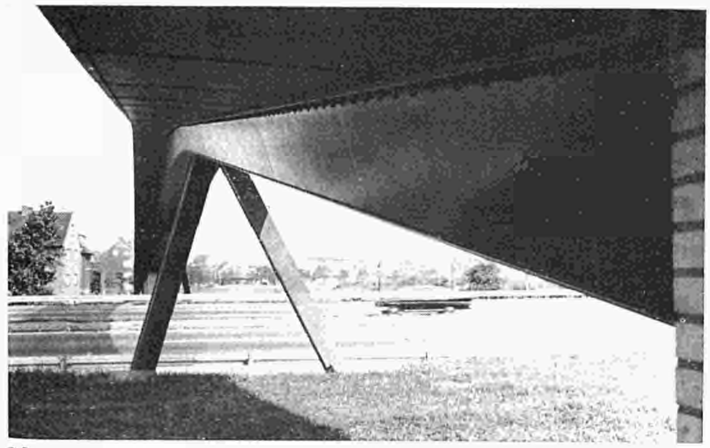

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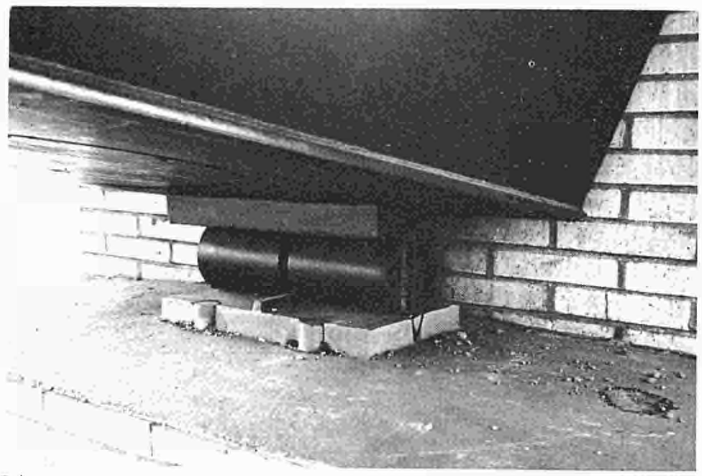

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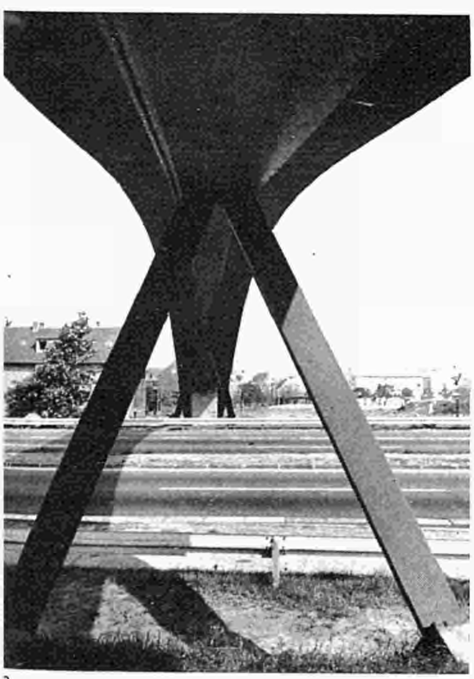

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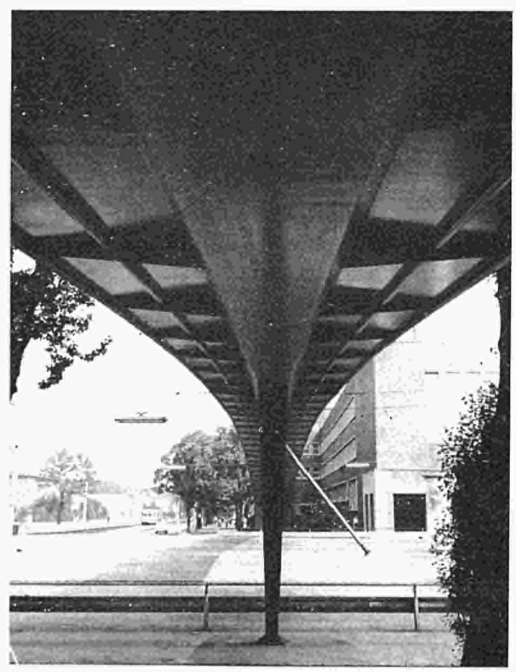

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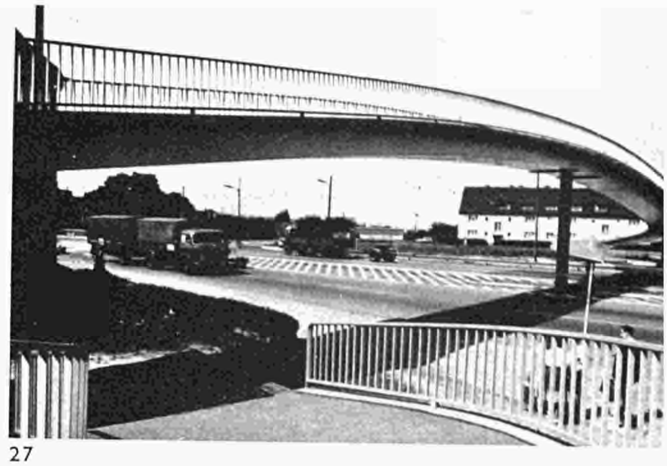
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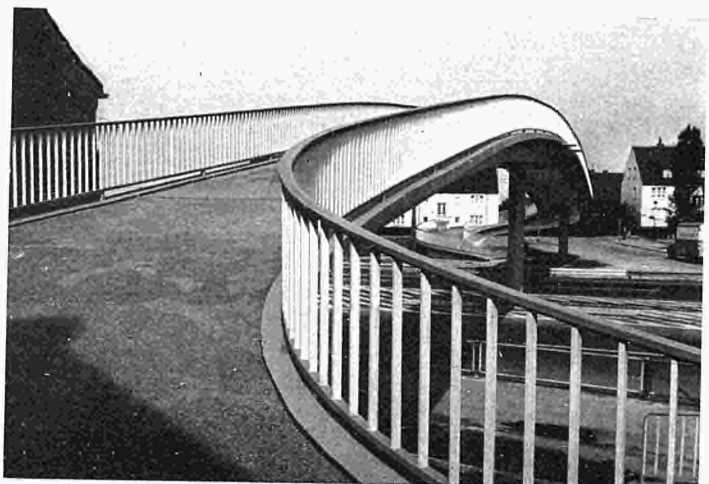
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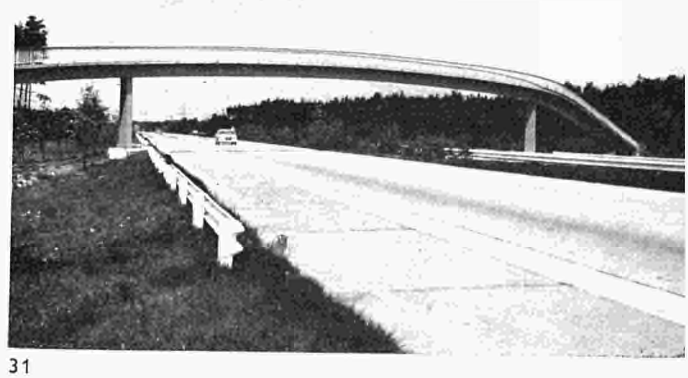
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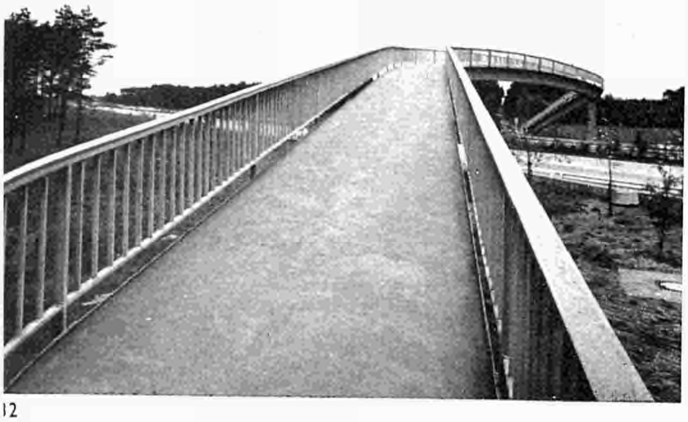

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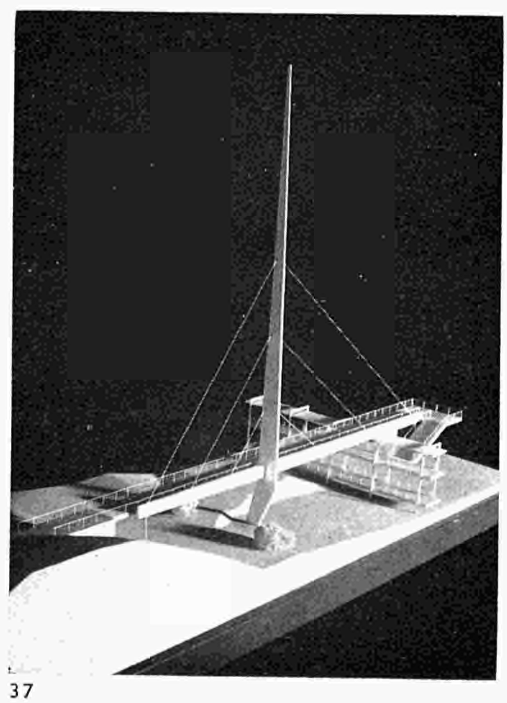

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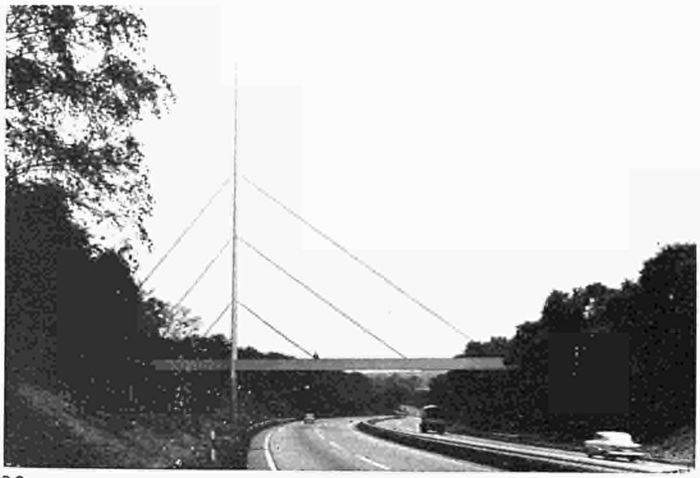
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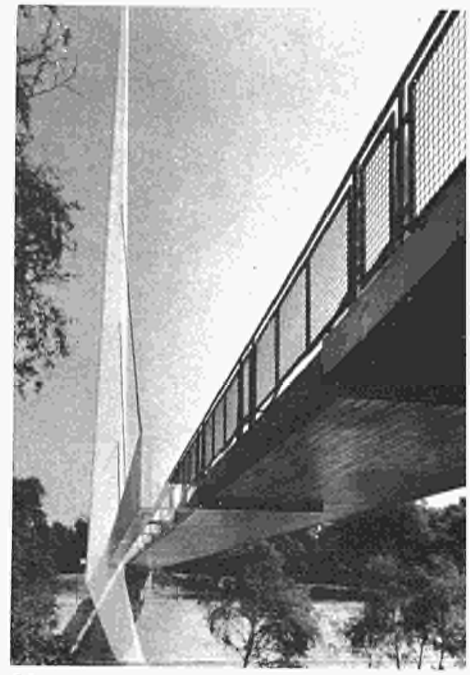
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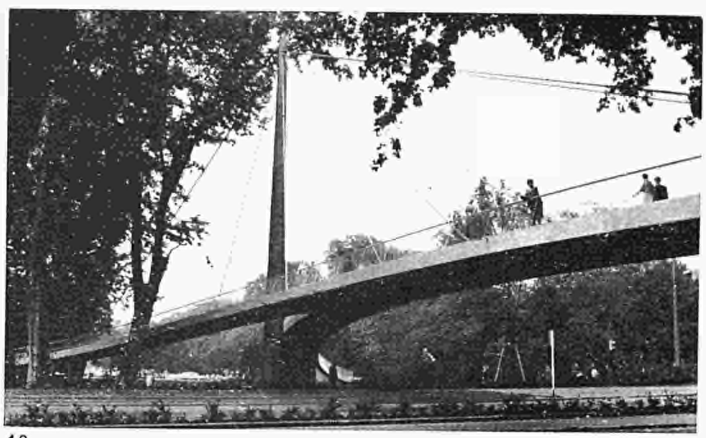
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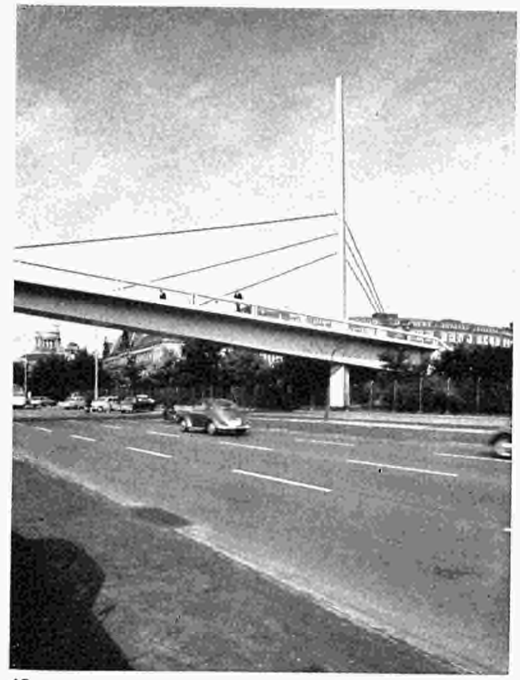
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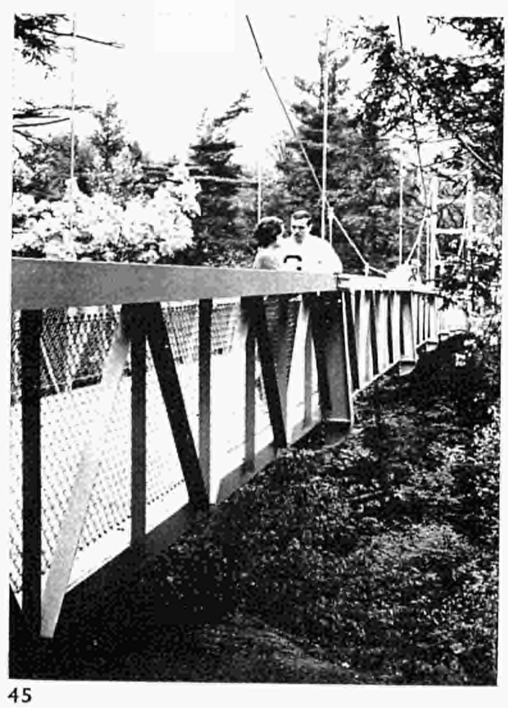
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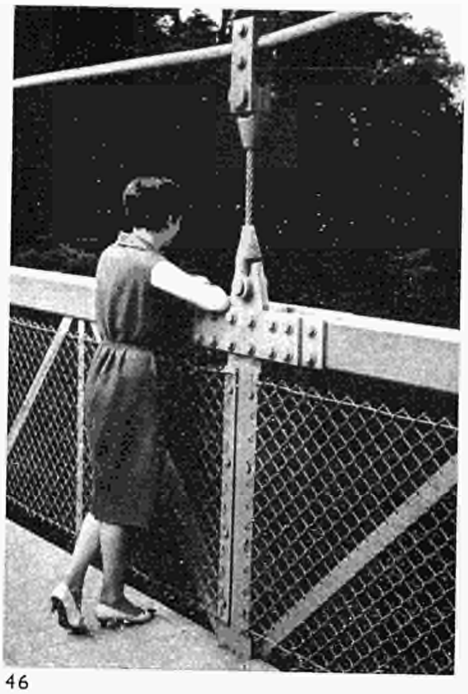
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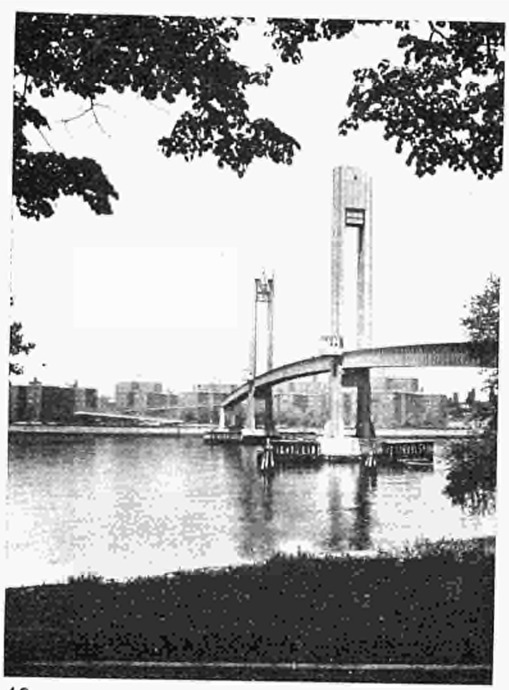
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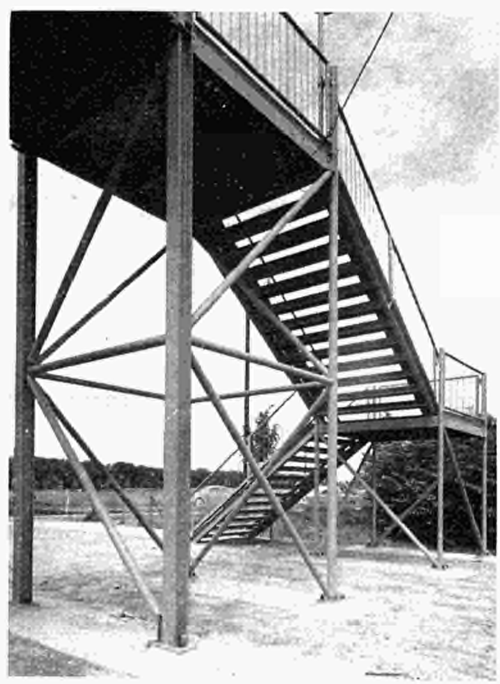

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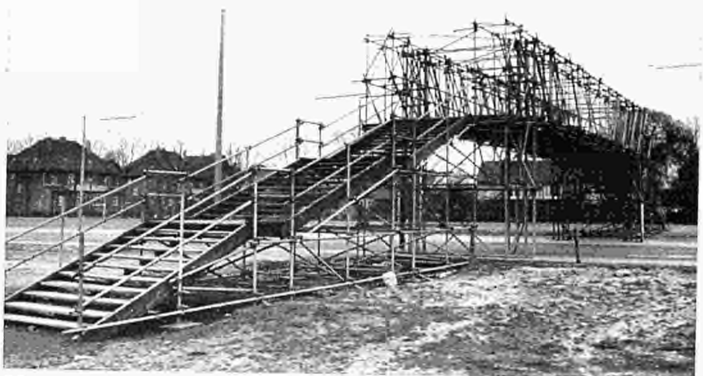
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Saverio SCHULTHEIS BRANDI

Road-Safety Devices

(Original text: Italian)

Introduction

The subject I have chosen, "Road-Safety Devices," deals with the many devices which are considered to be accessories in road construction *i.e.* guide posts, guard-rails, road signs, lamp-posts, screens, and so on. These are minor devices, of course, and, in the opinion of some, of secondary importance in comparison with this major review of the steel constructions which is to-day reaching its close here in Luxembourg.

However, inasmuch as these devices help to reduce the ever growing danger of road communications, they deserve quite a different evaluation.

Road communications, which are now more than a factor of the economy of nations, are becoming an integral part of the modern way of life. "More than a factor in the economy of the U.S. highway transportation has become a part of the American way of life" : these are the opening words of the report on the constitution of the important American Research Institute, and the Highway Research Board, which is a Division of the National Research Council.

The importance of reducing the danger of these vital means of communications is such that, when in 1946 the need was felt in the U.S.A. for a Committee for Road Safety, the Chairmanship was taken by the President himself : "President's Committee for Traffic Safety".

The dangers of road traffic are proved by the following figures : in the 13 European nations having the largest traffic density, about 100,000 people died and 3,000,000 were injured in 1963 through road accidents. These are frightening figures, and they are bound to grow every year because road traffic and especially the speed of vehicles are constantly increasing.

Consequently, if I quote from reliable statistics to show that road-safety devices have the effect of reducing the danger in a measure comprising between 20% and 40%, and if I further indicate that these devices are nearly always in steel, I shall also have shown consequently the importance of this branch of steel construction.

My exposition shall consist of the following :

A — The vertical devices, dealing with
- — guide posts,
- — guard-rails,
- — road signs.

B — Lightning, with special reference to the supports of the lighting units.

C — Screens with reference to their main protective effect, namely:
- — anti-dazzle screens,
- — wind-screens,
- — snow screens,
- — screens against falling rocks,
- — protection screens for pedestrian traffic.

The vertical devices

The vertical devices, already considered a simple integration of the horizontal devices, are acquiring ever increasing importance as the speed of motor vehicles increases.

The higher speed requires :

(a) the need to see, even with poor visibility, the full road line demarcated in a clear manner for an even longer section,

(b) the need to receive at an even greater distance the instructions of the vertical devices.

The vertical devices have consequently two distinct duties in relation to the above two items.

The first duty can also be called "visual aid to driving". Now that the obsolete kerb-stones and pillars have been discarded, this duty is to-day entrusted to guide posts and guard-rails. The guide posts have only the object of demarcating the roadway; the guard-rails add to this function mechanical protection, that is the prevention of vehicles of unintentionnally leaving the roadway.

The second duty, is of transmitting in adequate time a useful instruction to the driver, and is left to the road signs.

Summing up, the works to be examined in the field of the vertical devices are :

- — guide posts,
- — the guard-rails,
- — the road signs.

The guide posts

In view of the fact that for all vertical devices, international standardization is required, and that for these guide posts the vertical devices regulations issued under the name of "HLB" by the Federal German Traffic Ministry, contain detailed regulations, I think it advisable to deal with them.

The German regulation steel post (1) has a height of 105 cm. above ground and a polygonal section with two visible surfaces forming an angle of 30 degrees. It carries on the two faces, 30 cm. away from the top end, signals which vary according to the day or night visibility, and also for the right or left hand of the carriageway. The latter detail has the object of facilitating the orientation of the motor vehicle on a bend when visibility is poor. These posts are fixed to the ground by the amount necessary to withstand uprooting, but not to resist the impact on being struck by motor vehicles. It is this point especially which proves these steel posts to be superior to those constructed of any other material.

The steel posts are constructed in two elements : a lower element fixed to the ground and a top element which is the post proper, is connected by bolts to the lower element; the bolts are also of such size that they will shear on the lightest impact (2).

The posts are of ultra-light construction, weighing only 4 kg., and consequently offer very small resistance to the impact of motor vehicles (3).

Other advantages of steel post are, that they are easy to transport and to install, easy to replace when damaged as the base element is always fixed to the ground.

One last point to be mentioned at critical places, as in the so-called traffic islands, heavier posts are installed, also made in steel, and fitted with internal lighting equipment (4).

Guard-rails

Guard-rails are mentioned in this review as belonging to vertical devices, that is to say, the same category as the guide posts and the road signs.

This however, is doing the guard-rails a certain injustice, because, it it is true that they have the following two functions :

1. visual guidance,
2. mechanical protection,

the latter entitles them to be regarded as the most important among the road-safety devices.

The most important, but also the most debatable in fact, is that the function of the guard-rail is more complicated than it appears to be and experts are still in disagreement with regard to both its installation and design.

The question is so complex and controversial that it cannot be gone into adequately at this stage : I can only touch briefly on some of the more important aspects, notably :

(a) the function, or rather functions, of the guard-rail;

(b) types and features of guard-rails;

(c) the conflicting nature of road-accident statistics with reference to guard-rails.

The function, or rather, functions of the guard-rail

I have already referred to the two functions of visual guidance and mechanical protection and their respective importance.

With regard to the visual aid for driving, which has also psychological reactions, there is little to be said. To-day the tendency is towards increasing the visibility of the guard-rails, in dangerous sections, with painted zebra patterns (5 and 6). For night visibility the use of reflectors is becoming widespread (7).

With regard to the mechanical protection function which is defined as "a protection of the vehicle from sudden change of course with the danger of leaving the carriageway and which should be such that the vehicle should be brought back to a direction almost parallel to the guard-rail and with a decrease in speed that can be tolerated by the passengers," I will mention that the problems of the dynamics concerning the vehicle-guard-rail impact are attracting the increasing attention of research workers.

This is as it should be.

The danger of leaving the carriageway on motorways and especially of leaving the road at the left-hand side, and going over the partition verge (central reservation) with consequent possible head-on collision, has increased together with the speed of vehicles, whilst the means of defence, the guard-rail, has not been altered to meet the new demands, but has remained substantially unchanged and static for many years. To speak about present-day studies and even touch upon the dynamical problems involved is a matter which might lead us too far from the point. I should like, however, to mention :

— The work of the Road Research Laboratory,
— The work of Prof. Stahel, former Director of the Institute of Road Constructions at the Zurich Polytechnic.

Prof. Stahel's studies are of particular interest because they outline in the end, even though theoretically, the features of the new guard-rail, which should fully discharge the double functions of mechanical guide and braking effect.

The author sets the problem by analyzing two distinct effects of the guard-rail :

— A mechanical guide effect which is confined to bring the vehicle back to its correct driving course, allowing it to continue its run without appreciable variation in speed and consequently with practically a minimum, energy absorption, and
— A braking effect which takes away, wholly or partially, kinetic energy from the vehicle, through heat transfer or dissipation, off-setting the whole initial speed, perhaps vectorial, or at least part of it.

In conclusion, the author states that this dual mechanical action could be ideally achieved by a continuous and relatively rigid metal band connected to its supports by immediate flexible, but non-elastic elements.

Types and features of guard-rails

Types which have remained unchanged and are as follows :

— The rigid type, which is the guard-rail constructed in reinforced concrete,
— The semi-elastic type, namely, the guard-rail with a band of shaped steel sheeting,
— The elastic type, namely, the guard-rail consisting of a net with steel ropes.

As regards the rigid type, there are no novelties involved in the matter of applications, but a great deal of experimental research which will soon produce, perhaps some worth-while results.

As regards the semi-elastic type, I should like to mention :

(a) That the great variety of existing sections and thicknesses of the steel band is tending to be reduced. Sections are becoming standardised as the two types shown in figure 1, and thicknesses are becoming standardised at 3 mm. Greater thicknesses, like that of the Swedish section of 6 mm., have not met with favour;

(b) The grades of steel higher than St 37 are not considered suitable, at least in Europe. Grades St 52 and St 79, are used in American and Austrian sections, and give without doubt a higher strength, but what is most important here is the possibility of deformation. Moreover, the harder steels have a springy effect, which is far from desirable because it assists the "billiard" effect;

(c) There have apparently been some improvements in the joints near the supports. These joints are made preferably by overlapping (8 and 9).

I shall recall that the better class joints guarantee the continuity of the band, which is very important, because it permits the absorption of a greater amount of impact energy by a larger number of supports, thus transmitting it to the ground (10).

(a) Still on the matter of joints, the manufacture of guard-rails by press stamping, as preferred in Germany, has the advantage of providing the parts to be overlapped with a shape whereby sliding surfaces are produced;

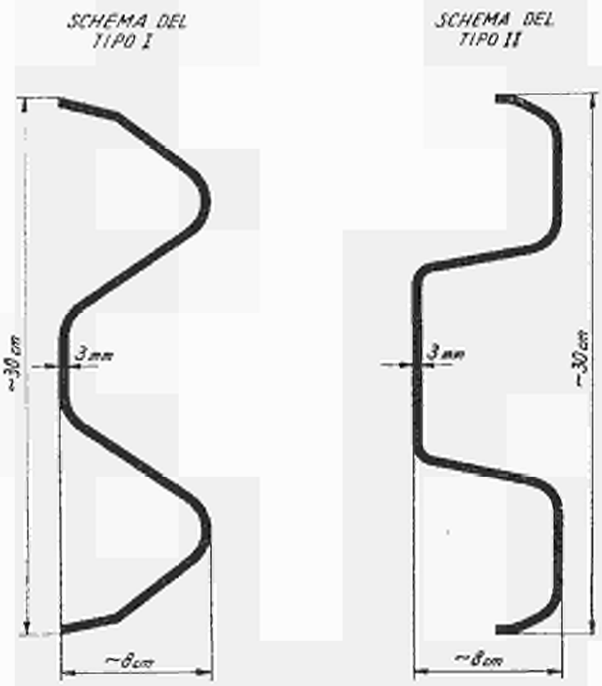


Fig. 1

(b) According to a new method already applied in the U.S.A., the bands are fixed to the supports with the application of spacers (11).

The spacers avoid the danger that the wheels of the vehicle, whilst sliding on the deformed guard-rail, should collide against the supports, which would make the accident more serious.

This type of guard-rail with spacers was considered, as far back as 1960, "a positive barrier" by the Highway Research Board, who, following a long series of tests, considered it worthy — in comparison with not less than 15 types examined — to be experimentally installed on the American motorways and confirmed their favourable opinion after two years' tests. In the last few years, great importance has been attached to the problem of avoiding the danger of terminal sections which may cause the serious accident called "cutting in". In addition to the solutions currently adopted (12, 13 and 14), there is a new one which has been the object of many experiments by the Highway Research Board and consists in gradually reducing to zero the terminal section (Fig. 2).

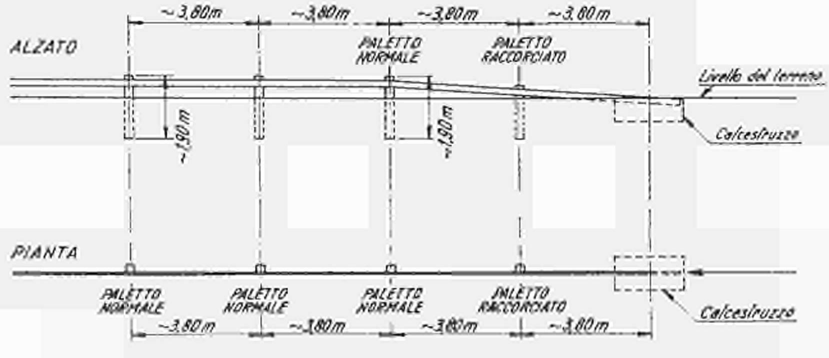


Fig. 2

Lastly, the guard-rail of elastic type (net with steel ropes).

On the strength of its behaviour from the point of view of its braking effect, this type of guard-rail (Fig. 3) has been rather well received in the U.S.A., where in 1962 about three were installed for every ten of the semi-rigid type. It has, however, some serious drawbacks : there is the possibility of injury to the occupants of open vehicles, and especially to motor cyclists. Its maintenance costs are high, and, in particular, it is liable to be displaced some distance upon impact with considerable deflection, which is completely unacceptable with a central reservation of limited width.

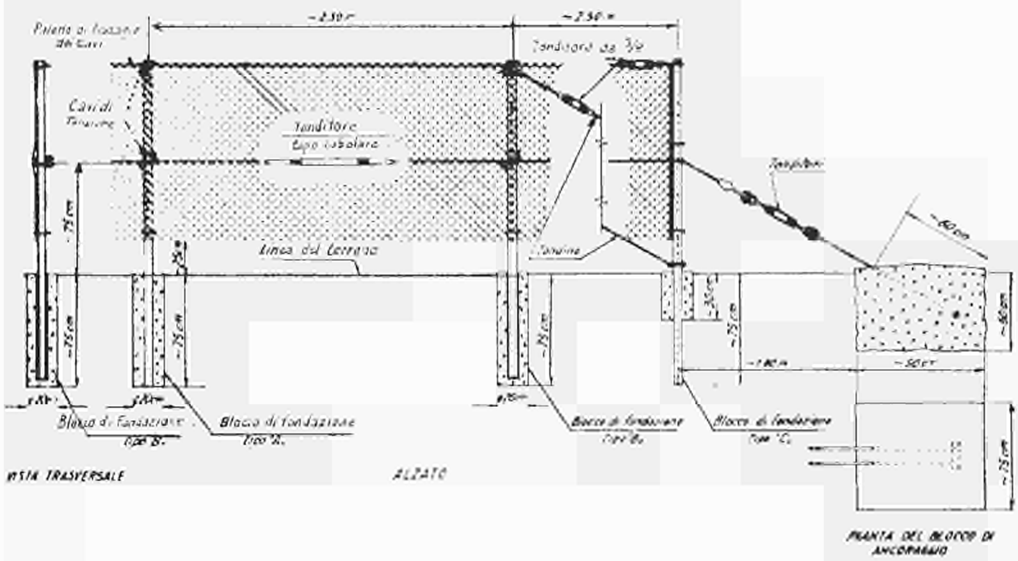


Fig. 3

Conflicting nature of road-accident statistics with reference to guard-rails

The technical press has recently reported contradictions in the statistics of road accidents carried out, after the guard-rails were installed, with special reference to the guard-rails of the partition verge (central reservation).

The contradictions mentioned are between American (U.S.A.) and European (German) figures.

The following points indicate, however, that the contradictions are more apparent than definite.

(1) Road-accident statistics are the most difficult to compile. To "polish" and make uniform the results is a hard job requiring the strict application of the principle of statistical science in view of the complexity of the mechanics of road accidents. For this reason, few statistics are properly intercomparable.

(2) The geometrical features of the road have a considerable bearing on accidents. In the United States the "Safety cross-section" is the main preoccupation of the organizations handling the construction of motorways. There is a great difference in typical cross-sections between American and European motorways : partition verges 10 to 20 metres wide in the U.S.A. against 4.50 to 5.60 metres, with some exceptions,

on the European motorways. Also very slight gradients in slopes in America, which is not always found on the European motorways.

(3) According to recent investigations by the Highway Research Board, serious head-on collisions following crossing of the partition verge were eliminated when guard-rails were installed. On the other hand, the total number of accidents seems to have increased. This, however, does not mean much if only the number and not the seriousness of the accidents is reported, since it is well known that minor accidents can easily give a high percentage. What is important is that head-on collisions have been eliminated by having guard-rails on the central reservation.

(4) The recent statistics compiled in Germany afford reliable evidence. The figures refer to 1962 and concern 2,000 cases, reported on a standardized form, of vehicles colliding with the guard-rail following over-running of the central reservation.

These are the figures :

(a) 97.1% of motor cars and 90% of commercial vehicles were held by the guard-rail;

(b) 1.2% of the motor cars and 5.7% of the commercial vehicles overran and smashed the guard-rail, but stopped on the central reservation;

(c) 1.7% of the motor cars and 4.2% of the commercial vehicles smashed the guard-rail and entered the opposite carriageway.

These data were supplied by an authoritative source, Engineer Lapierre, consultant ot the German Federal Traffic Ministry, Division of Road Constructions.

If we now compare these data with the following well-known data made public by Prof. Bitzl (in an interesting and widely distributed publication) with regard to the behaviour of motor vehicles which on the same motorways were overrunning the central reservation not then provided with guard-rails, *viz.*,

— vehicles left on the central reservation, 44%;

— vehicles gone over to the opposite carriageway, 56%;

no doubt can remain as to the great contribution which guard-rails have made to road safety.

It must be added that the German statistics given refer to sections of motorways equipped almost exclusively with steel guard-rails.

Road signs

The other function of the vertical devices, that of giving in good time adequate instructions to the driver, is also very important for road safety. For instance, any uncertainty or hesitation on the part of the driver concerning the direction to take when approaching cross-roads may cause serious accidents and, in any case, cause a dangerous slowing down of the normal traffic flow.

As I have already stated, this function is entrusted to the road signs, and the rapidity with which the message is given is evidently a function of the speed of the motor vehicle.

As speeds increase, most road signs, especially on motorways, must be visible not at 100 or 200 metres away as in the past, but 300 metres away, and, in certain cases, even this is not enough.

This has given rise to a dual problem :

— of constructing road signs of larger size,

— of fixing them in a position that they can be seen from far away, namely, no longer placed by the carriageway but overhead on the carriageway.

To solve both problems, it is preferable to have recourse to construction in steel for the following reasons:

a) The road signs in light-gauge steel sheeting enjoy the known advantages of this material for the following reasons :

— its high mechanical strength, coupled with light weight,
— good resistance to corrosion in the galvanized type,
— a very good base suitable for enamelling, and
— lends itself very well to pre-fabrication processes.

With regard to the last item, I wish to point out that manufacture of road signs from prefabricated components is finding ever increasing demand because this system offers obvious advantages, namely economy, speed of erection and ease of replacement (15).

The elements are connected to one another and to the supports by bolts or special clips and the erection can be carried out either by completing the whole panel *in situ* or by erecting it, one element after the other, directly on the support.

b) The supports of the road signs are becoming an important function requiring light, yet sturdy structures — two properties which steel is eminently fitted to provide. From simple modest supports on the road sides, consisting of a single post, we have gone over to supports still on the side of the carriageway, but provided with long brackets (16), which "hang" the sign over a part of the carriageway, and finally we have gone over to supports crossing the road overhead, namely the so-called portal supports.

These portal supports are available in a wide range of types, from the lighter models, tubular (17 and 18) or box section (19) to the more imposing structures with service gangway, or either tubular (20 and 21) or box section (22 and 23).

Lighting

The importance of road lighting from the point of view of traffic safety is proved by

— the number (and seriousness) of night accidents, as compared with day accidents, in unlighted roads;
— the reduction in the number (and seriousness) of night accidents after lighting equipment was installed.

In this connection, it is interesting to note comparative statistics, called "day-night" and "before-after", of the accidents.

— With regard to the first (day-night comparison) it appears that with a night traffic equivalent to a third of the total, the night accidents amount to 40% of the accidents in the 24 hours, and this percentage goes up to 50% if we take into account fatal accidents only. This is according to the findings of the Highway Research Board on some roads of the State of Indiana.
— With regard to the second (before-after comparison) it appears that the reduction in the number of night road accidents due to lighting is about 30%; this is according to the findings of the Road Research Laboratory on some roads in the London area.

It is therefore natural that throughout the world large works of road lighting are being utilized or envisaged. I give as an exemple Belgium, where they expect to provide lighting for 700 km. of roads.
Where does steel fit into the installation of up-to-date road lighting?

It comes in very prominently, since steel is the best material for making the supporting columns for the actual lights. To demonstrate the greater advantages of steel supports, I need only run over the following points :

(1) What should be, in a modern road lighting system, the features of the light sources, especially as regards their positioning.

(2) What should consequently be the features of the light source supports (consequently, typical supports).

(3) What are the reasons why the steel support represents in a perfect way the typical support.

The requirements of the light source

Some basic pinciples of road light engineering concerning especially :

— the light output on the road surface (minimum 2 candles per square metre),
— the uniformity of light output in a direction perpendicular to the axis of the road (minimum value 0.4),
— the reduction in dazzle according to the two systems called "cut-off" (favoured on the Continent) and "non-cut-off" (favoured in Britain and in America) and to the recent compromise system called "non-cut-off at an average angle" (sudden drop in luminosity between 81 and 86 sexagesimal degrees),

determine the requirements of the light source as follows :

(a) height = from 8 to 12 metres on very busy roads, *i.e.* arterial highways,
= about 30 metres in squares, cross-roads and the like,

(b) spacing of lights in a longitudinal direction in straight sections equal to from 3 to 3 1/2 times the height; on bends, slightly lower,

(c) overhang of lanterns (namely, distance between their vertical line and the edge of the roadway) : from 1.00 to 2.50 metres,

(d) inclination towards the horizontal line : from 5 to 20 sexagesimal degrees,

(e) type of lamp : preferably fluorescent or mercury vapour type, or a combination of sodium vapour and fluorescent light, which has been adopted in Germany and Switzerland.

The requirements of the light-source column and definition of the typical support

In view of the need for supporting the types of light source mentioned in the above positions, the typical support of the light source is defined by the following features :

(a) height : obviously the same as in the case of the light source;

(b) overhang : from 1.50 to 3.50 metres. The latter dimension is higher than that described for the light source because account is to be taken of the fact that the column cannot be placed on the edge of the carriageway, but must be at a certain distance from it;

(c) inclination of the end of the column where the lamp is attached : same as for the light source;

(d) hollow space in pillar : sufficiently large to house the electricity supply equipment, which is sometimes complex.

The typical support designed in steel

In view of the fact that the typical support is as above defined, there is justification for the statement which I have previously made that steel is the only material in which it can be produced.

The recent progress achieved in steel grades and in tube manufacturing processes has led to the fact that the steel support, which was of tubular type for some time, has reached such a perfection from the technical point of view and from that of appearance that competition from supports made of other material need no longer be feared.

The steel tubular pole, whether made from standard tapered or conical tubes or from thin-gauge sheeting with an average thickness of 4 mm., shaped in various sections, has now such a perfection of shape that it could only exist as an expression of technical perfection.

The steel tubular pole stands up straight, tapering gradually and gracefully bending at the top, "whip like", like the stem of a flower. The inclination of the whip provides the overhang and meets the requirements, of road-light engineering as regards the direction of the light beam from the lamp fixed to its end. The photo 24 shows the positively flowerlike grace of the now classical type of steel tubular column with whip end.

But quite apart from its pleasing appearance, it is the technical features of the tubular steel column that make it so ideal for the purpose. Thus,

— it allows considerable heights and the considerable overhangs required, to be achieved, whilst having a small section and of a light gauge, and at the same time being of light weight and slim. All this is due to its high mechanical properties;

— it permits easy fixing of the complex lighting equipment;

— it is hollow and can thus accommodate, at a conveniently accessible height, the electricity supply equipment;

— it is of comparatively light weight, which makes it easy to transport and to erect;

— it offers minimum resistance to the wind;

— its maintenance is inexpensive, since it is protected by a bituminous internal coating;

— in the case of extra-long pieces as for lighting columns in squares, it can be transported in several pieces which are then easily connected *in situ*;

— it is also possible to fix to it, by simple welding, brackets, plates, ladders, etc.;

— lastly, it can be designed so as to have special sections of minor strength where there may be a possibility of collision with motor vehicles, a requirement which might be enforced following the latest statistical data on road accidents compiled specially by the Road Research Laboratory, London.

Photos 25, 26 and 27 show a promising type of tubular column, which can be made with whip ends having varying lenghts according to the lighting required.

Photo 28 shows a column for a square, about 30 metres high.

Photo 29 shows a section of the dual-carriageway Hamburg-Frankfurt motorway, lighted from the central reservation by a single row of double whip columns.

Photo 30 is a fine example of lighting from the central reservation with a double row of joined columns with single whip. It is a section of the Autoroute du Sud in France.

Screens

Lastly we must examine the set of devices which, although different from one another, have in common the function of protecting road users from outside effects of a various nature, which can somehow interfere with driving or obstruct the traffic.

These annoying effects are chiefly caused by

— headlight dazzling from motor vehicles going in the opposite direction;
— wind gusts;
— snow and snow avalanches;
— falling rocks;
— interference by pedestrian traffic.

The defence equipment used against the above actions is appropriately called "screens".

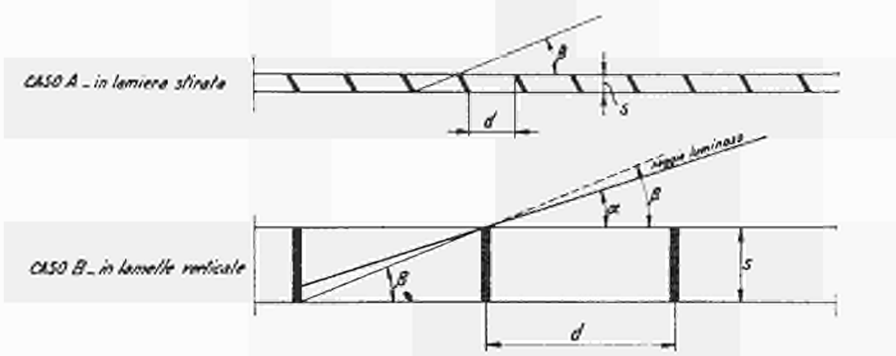
Anti-dazzle screens

A main course of danger with night traffic in unlighted roads is the dazzle from the headlights of vehicles moving in the opposise direction.

If the road is a dual-carriageway with central reservation, it is possible to eliminate this cause of accidents by installing suitable screening on the reservation.

The metal screens, in chequered plate or with vertical spaced laminations, do not have the well-known drawbacks of certain types of screens, such as hedges, bushes, plastic panels, etc. and consequently are preferable.

Their operation can be easily explained by simple geometrical considerations (Fig. 4).



α= semiapertura angolare del cono di luce del proiettore
β= angolo di schermatura
α<β condizione di schermatura

Fig. 4

Les us call α the semi-angular aperture of the light cone of the projector and β the screening angle, defined as the minimum angle, relatively to the plans of the screening panel and consequently relatively to the road axis of the ray passing between the two adjacent elements of the screen. It is evident that if β is greater than α (more accurately than α increased by the angular tolerance between the axis of the projector and the axis of the vehicle) no ray from the projector will pass through. The values "s" and "d" in the illustration, which are characteristics of the screen, determine the angle β, which, (as angle α is fixed by the highway regulations), practically must, be slightly greater than 20 sexagesimal degrees, for the straight road sections.

The requirements of a good anti-dazzle screen are as follows:

— it must allow by day good vision of the other side of the carriageway;
— it must allow the passage of snow;
— it must not be a danger in the case of road accidents;
— it must allow, at reasonable intervals, passage for the service staff and the Road Police.

Metal screens meet all these requirements excellently.

The types with vertical laminations offer several advantages over drawn sheet; the snow passes more easily, and the angle of the laminations can be adjasted, which is important because the screening angle β must not be the same on straight sections as on bends.

The drawn-sheet screen is shown in photos 31 and 32. Photos 33 and 34 show two side views of another screen also with vertical laminations, but of a different shape.

Wind screens

The wind, especially if accompanied by side gusts, has an influence on motor vehicles which increases in proportion to the speed of the vehicles. As speeds are constantly increasing, accidents due to the wind, which cause the vehicles to leave the carriageway, are ever more frequent. Fortunately, the road sections which present this danger can be easily identified: *i.e.* woods, embankments, when leaving underpasses, tunnels, bridges, etc.

The best protection, (apart from screens consisting of plants and hedges — which can be inefficient and dangerous as anti-dazzling screens) consist in this case as well of steel screens, which have a special appearance as shown in photo 35.

These screens are made up of a set of grilles each consisting of light-gauge metal strips woven into a wire net fixed to a light-weight frame. Its principal advantage is the extreme ease with which it can be transported and installed.

Snow screens

The invasion of the carriageway by the snow causes serious trouble to the traffic and becomes a grave danger when the invasion takes place suddenly, as in the case of snowdrifts.

The gradual accumulation of snow takes place especially at given points and it is therefore possible to prepare suitable protective installations.

Experience has proved that snow screens are more efficient if they are not of solid panel type but let the wind pass through; in fact, the optimum consists of a screen in which the ratio full/open is 50/50.

Photo 36 shows one of these screens consisting of a steel rod frame with wide mesh into which are woven light-gauge metal strips approximately 15 cm. wide. The screen is of very light weight and is firmly fixed in the ground, being fitted at the sides with two right angle section legs.

These screens can be easily transported over mountain regions where they are particularly required.

Against snow drifts, however, much stronger screens are reauired (37). They have uprights in double T sections and cross-members in profiled plate; the various components are then nearly always connected by bolts to facilitate transport.

Screens against falling rocks

Another danger to road traffic consists of falling stones and rocks. This can happen in sections where the road is excavated over sliding ground and very frequently on mountain roads. The screens used in this connection are very similar to those against snow drifts.

Also in this case, the screens have uprights in double T sections and cross-members in fabricated plate, whilst their legs are provided with extension attachments so as to adapt them better to the gradient of the ground.

It is to be noted that they are always mobile devices, to be suitable for mountain roads, where the fall of rocks may take place at different points. Very often they are of highly complex in construction (38).

Pedestrian traffic screens

At certain critical points of the road, such as in the proximity of stadiums, schools, bus stops, etc., pedestrians are often apt to invade the carriageway. At those points, the two types of traffic, vehicular and pedestrian, must be rigidly kept apart by a fixed screen so as not to cause serious mutual damage.

This screening, commonly known as „fencing", is usually made in steel, which constitutes the most economical and elegant solution of the problem.

Photo 39 shows a fence near a bus stop. The fence consists of a metal net with a tubular frame.

Photo 40 shows a fence, also tubular but reinforced, alongside a narrow pavement by corrugated sheet panels.

Conclusion

The review of the works of steel in the service of road safety, a review which was necessarily brief in view of the short time allotted, can be concluded here.

Strictly speaking, it is not complete because, as correctly stated in a publication by O.T.U.A. "L'acier dans l'équipement de la route", we could also logically include as factors of road safety all the equipment which aids the motorist's comfort and the efficiency of the motor vehicle, namely, motorway cafés, motels, service stations, filling stations, etc.

Unfortunately, however, owing to the short time at my disposal, I was not able to refer even for a moment to these supplementary services, even though this means not mentioning other important applications of steel in road construction.

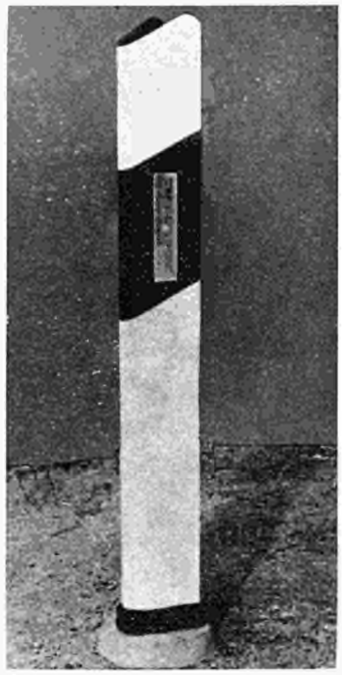
Summing up, in most of the roadway accessories examined, steel has been found to be the best material, only in a few cases are other materials rivalling steel able to compete.

Therefore, we could then calculate the amount of steel used in the construction of a road section, for works which are considered to have minor importance, but are always present on the roads. This calculation would give a tonnage which shows up favourably in comparison with the tonnage used, for the same road section, for larger, more showy, more „spectacular" works, such as bridges and flyovers.

This, calculation which for us technicians, who shall never become business traders, is of little interest.

We technicians are in fact interested in indicating the tremendous contribution which steel makes to road safety and, consequently, to the national and international traffic, as well as to the progress of civilisation.

It is my intention that this paper should contribute to the above purpose.



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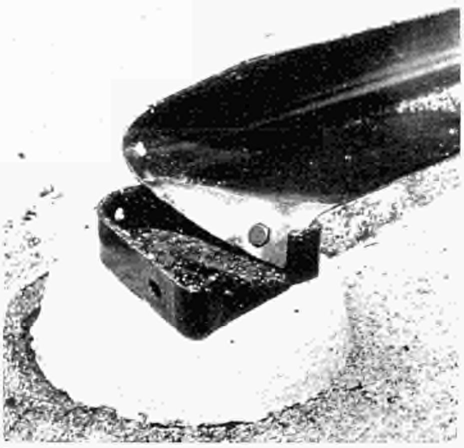

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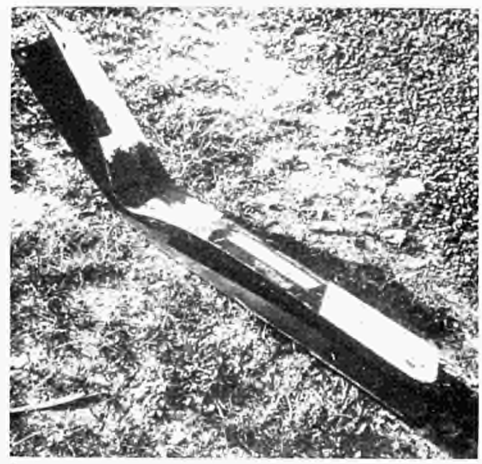

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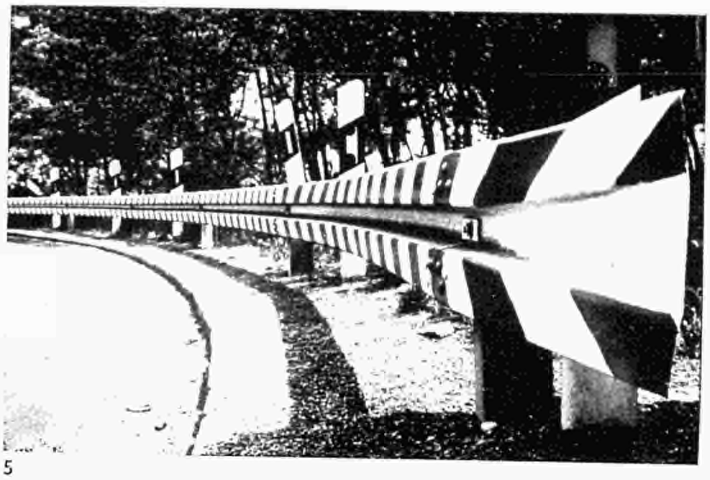
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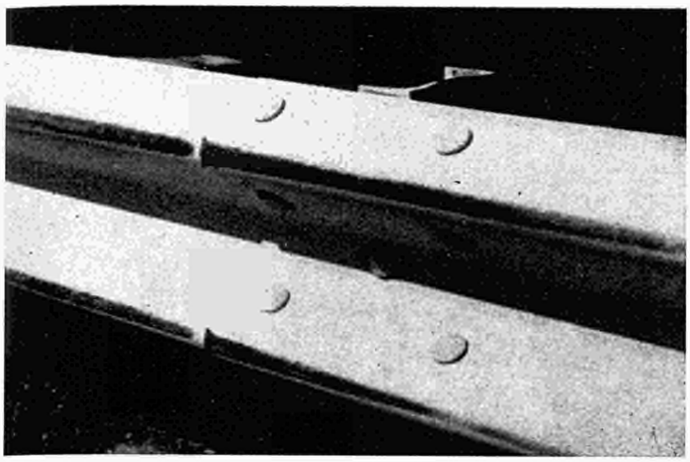

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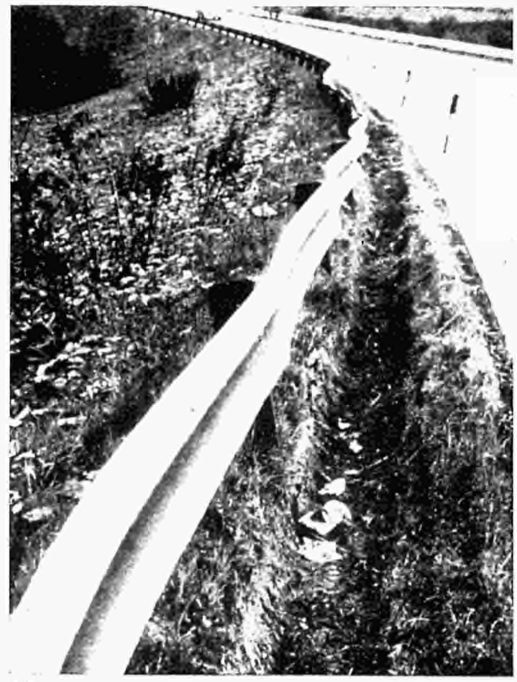

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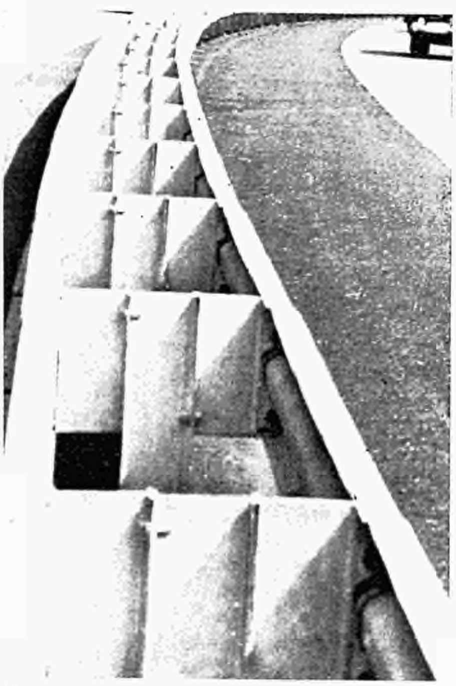

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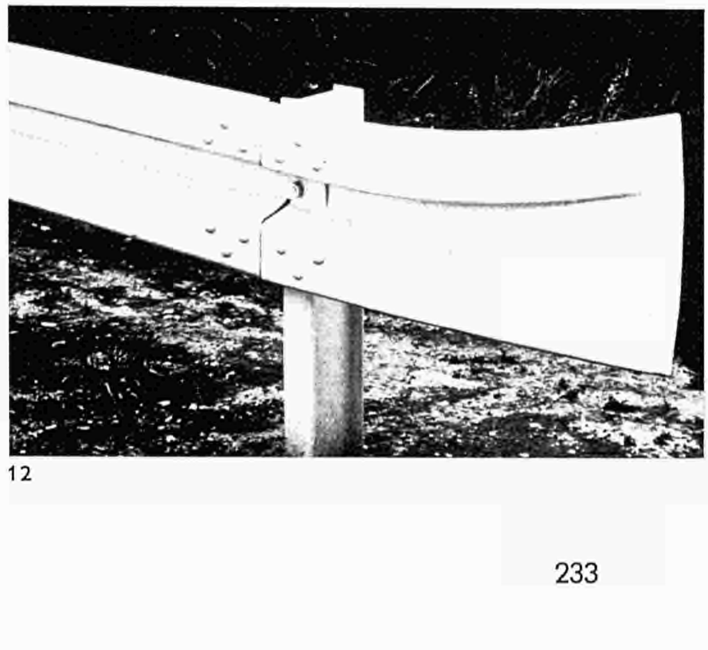

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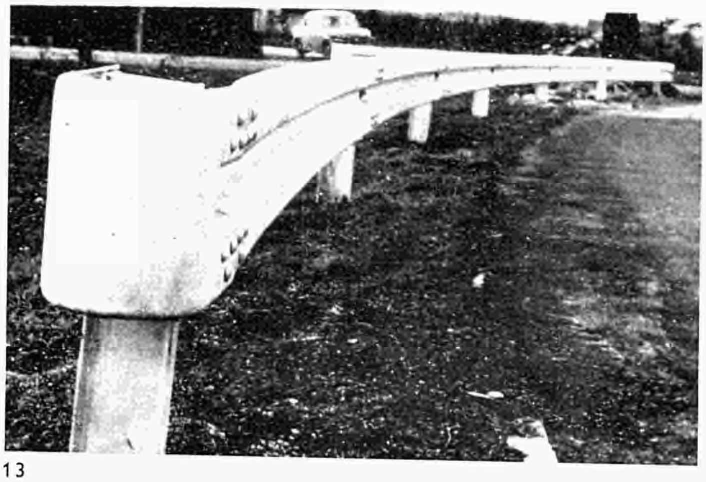
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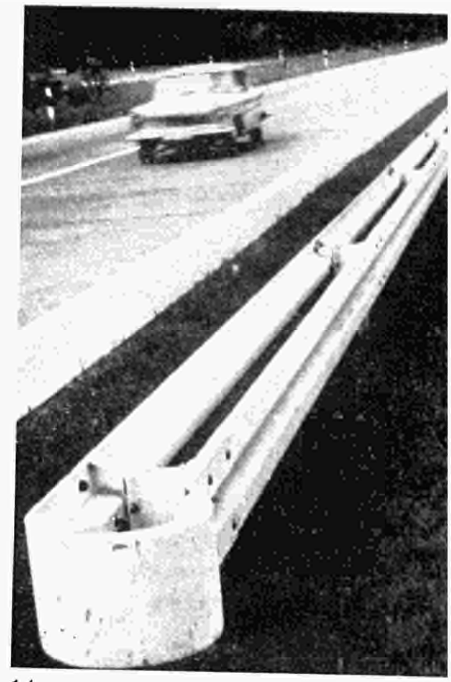
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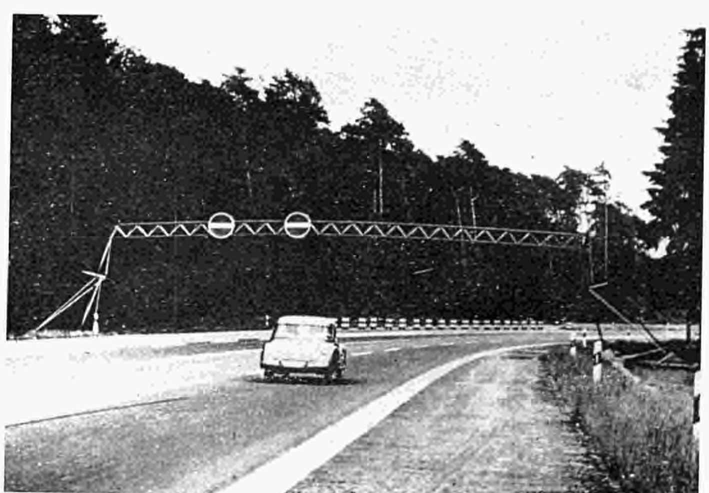
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ALT m 500
MILANO
18


1 Essen
Mülheim (Ruhr)
Hannover
Köln
Krefeld
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E11 Frankfurt
Mannheim
Karlsruhe
Basel
Heilbronn
Stuttgart-W/
Leonberg
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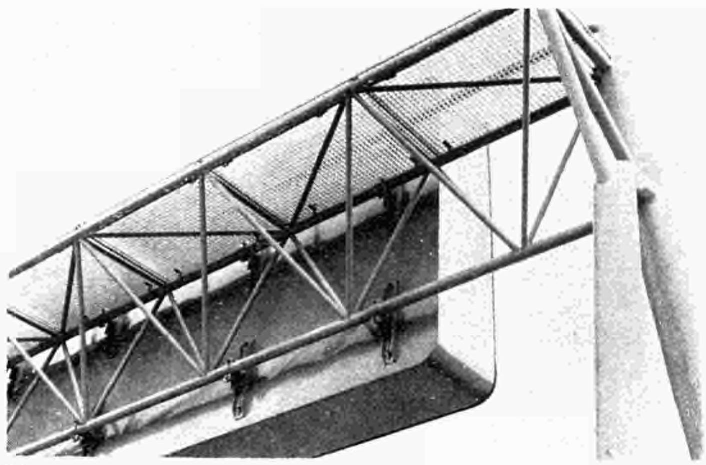
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Hannover E3
Dortmund
Emmerich E36
Oberhausen
Duisburg-Mitte
Venlo E3
Moers
Mülheim/Ruhr
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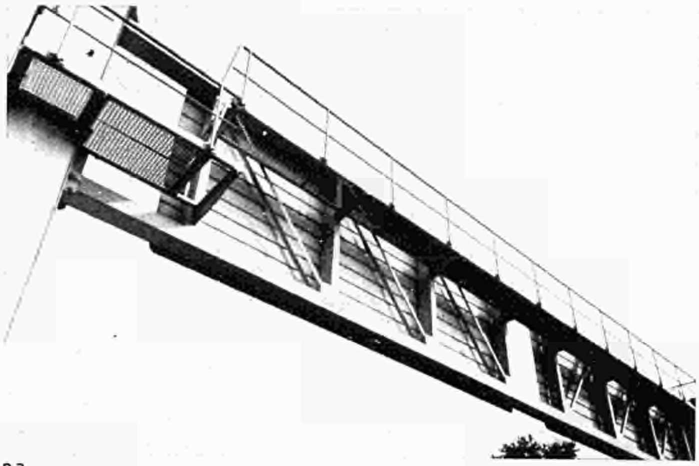
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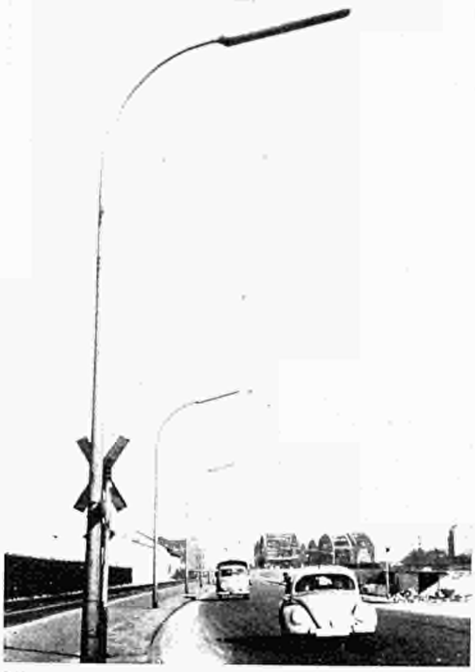
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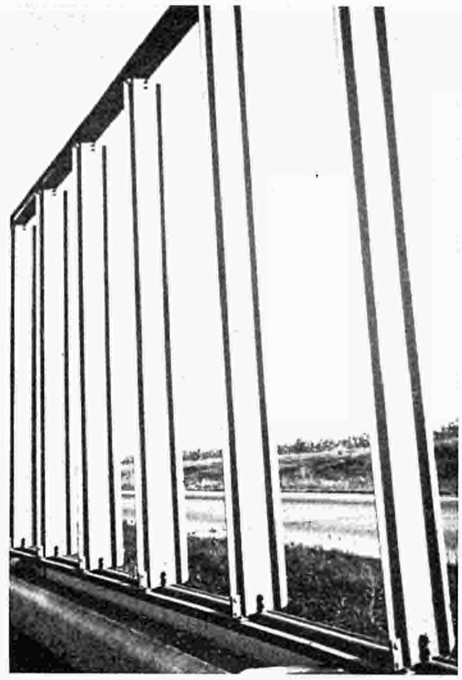
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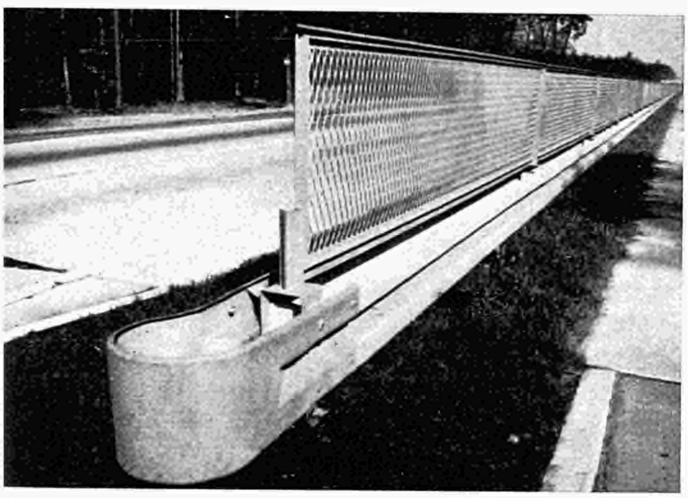
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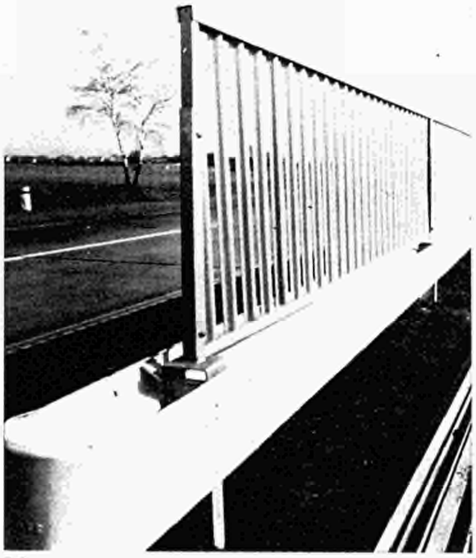
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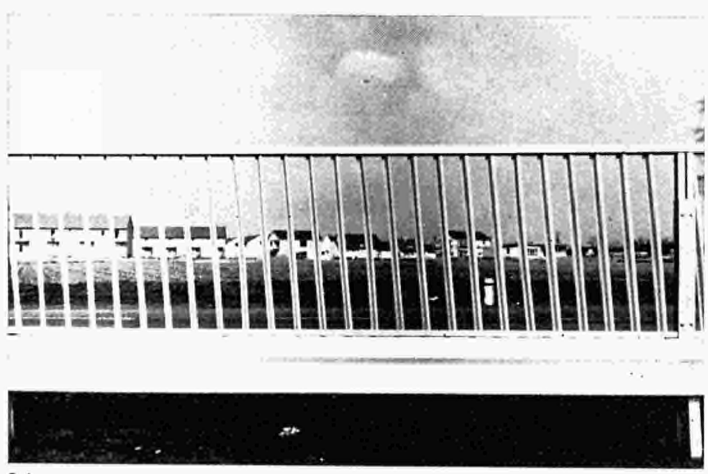
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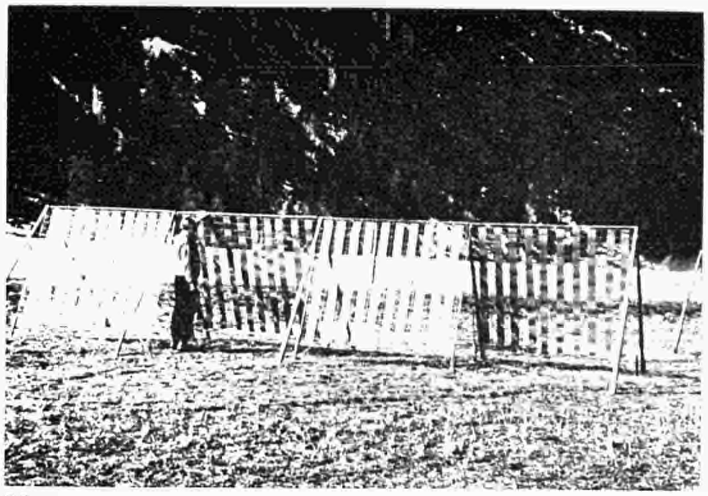
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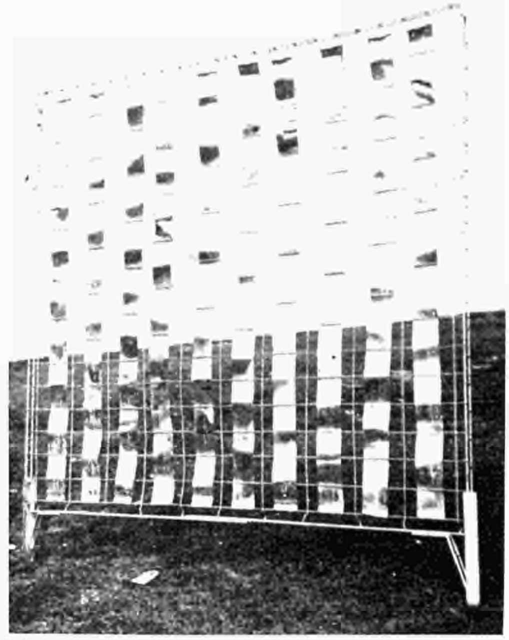
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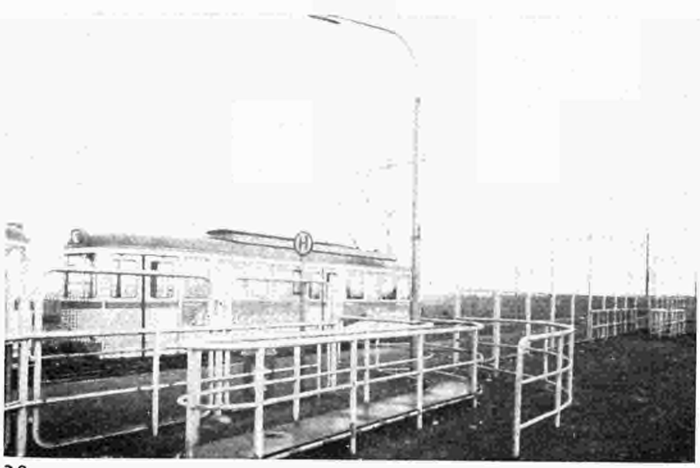
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Paul EIDAMSHAUS

Utilization of Cold-Formed Sections in Construction

(Original text: German)

Cold-formed sections are being used more and more as load-carrying members in modern steel design. These sections are cold-formed from steel strip to shapes for hollow stiffeners or longitudinal ribs for deck units for bridges and roadways. These ribs work compositely with the deck plate: Figure 1 shows a few typical sections.

Such cold-formed sections can be made, either by bending steel strip in a flanging machine or by a continuous rolling process in which the steel strip is fed from a coil through the driven shaping rolls of a rolling mill and then shaped to the appropriate profile.

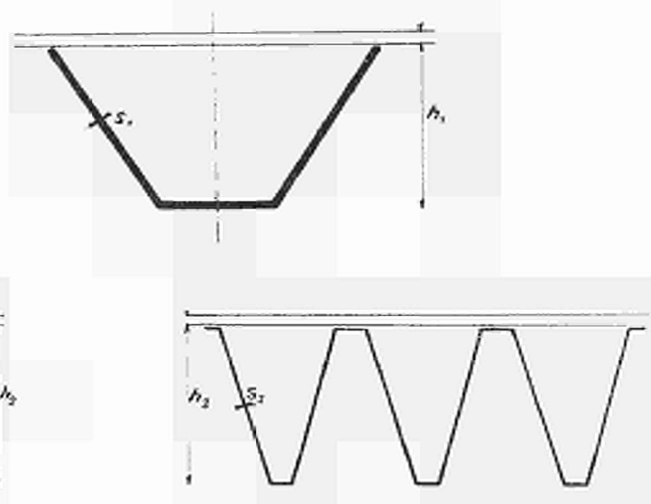


Fig. 1

Cold-rolled sections have certain advantages over the flanged sections. These will be listed as they help improve the economy of the structures under discussion :

(1) Cold-rolled sections can be produced in lengths to 50 ft. or more, and this, of course, reduces the numbers of splices in large deck units.

(2) Closer sectional tolerances than those generally possible with flanged sections cut down subsequent work.

(3) Cold-rolled sections allow the possibility, in a particular manufacturing process, of raising the yield point of the material — which occurs at points of deformation such as the corners — not only at the corners but also along the straight sections of the material so that higher stresses are permissible over the whole section. An example of this is a road-bridge on the outskirts of Hamburg (1 and 2).

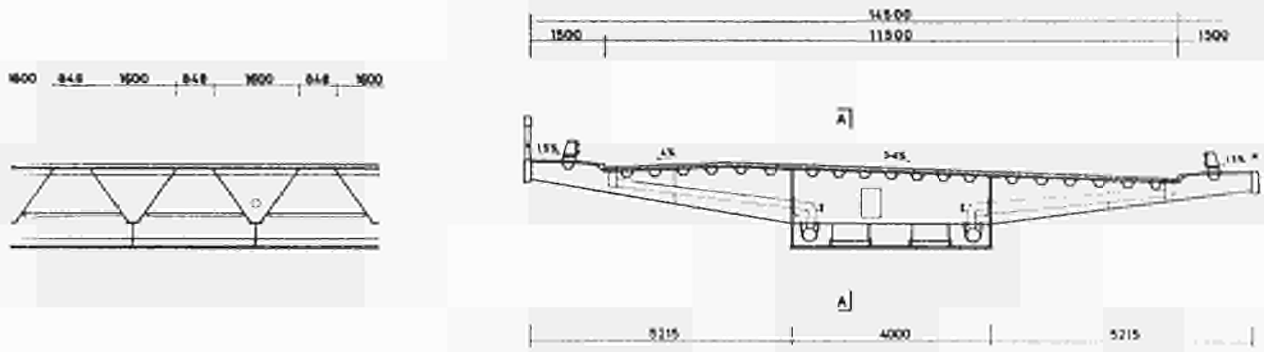


Fig. 2

Figure 2 shows the cross section and longitudinal sections of this bridge. The road deck and the longitudinal Jibs which are of particular interest can be clearly seen. In designing these Jibs, it was originally intended to use steel St. 52 on account of the high calculated stresses.

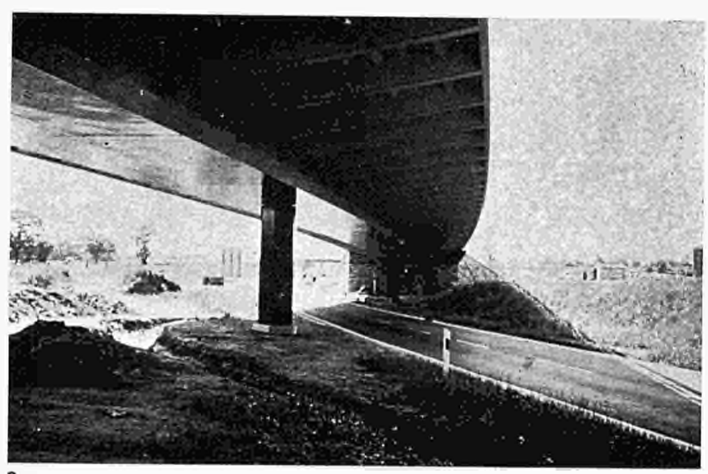
However, cold-rolled sections of St. 37, with a guaranteed yield point of at least 38 kg/mm². (24.13 t/sq.in.) were used and this relatively small improvement brought a worthwhile saving. It also happens that welding a St. 37 steel is easier than welding a St. 52 steel.

Consequently, the raising of the yield point in this type of cold rolling can effect an economy by either

a) changing to a cheaper grade of steel, which was done in this case, or

b) using a thinner section of the same quality steel, that is by saving weight.

Thanks to the open-minded attitude of the client (a public authority) and the constructional engineers responsible for the work, this roadbridge in Hamburg was the first in which this "cold strengthening" effect produced in cold rolling was made use of to allow high working stresses — a principle which in the interests of the steel industry, it is hoped, will find more frequent application.



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Jürgen DEMMIN

Culverts-Steel Water Pipes

(Original text: German)

In former days it was customary for bridges to be built to last for generations; nowadays we live in an age where structures, not yet thirty years old, have sometimes to be demolished because they are no longer able to cope with current road traffic demands or the requirements of waterways.

It would therefore be a waste of public money and of valuable building capacity if, instead of suitable structures with a predetermined life-span, structures were designed whose actual life exceeded their useful life.

The economic design of building works corresponding to their expected useful life has not yet been able to make its mark on the building world. While it is very difficult accurately to forecast the useful life, it can well be said that, apart from major civil engineering schemes, road works with a design life of more than 50 years will outlast their useful life.

To-day, on looking back, we find only very few road schemes which are more than 50 years old and which still meet present-day requirements.

Over the past few years it has been found that the development of road traffic has been not linear but progressive.

There is no reason to suppose that this trend will change in the future, and this being so it would be rash to try to forecast traffic growth over the next fifty years. Therefore, why should we erect structures, especially minor ones, which are dependent on the traffic growth, with a life-span of more than 50 years?

Consider, for example, the case of the Cologne-Leverkusen Motorway where a few years ago large concrete bridges, which were not even thirty years old, were demolished at great expense, in order to widen the motorway to six lanes; this confirms the argument and shows the advantage of types of construction which can not only be erected quickly and easily but can also be equally rapidly dismantled when the need arises.

The development of culverts stemmed from these ideas, — ideas, which are being put to you in this short report.

The first culverts were produced in the U.S.A. about seventy years ago for use as pipes for streams under road and railway embankments. Corrugated steel pipes were used, in various plate thickness to suit the loading (1 and 2).

The advantages of these culverts are their light weight, the exceptionally short installation time and their capacity for carrying full load immediately after being covered.

In addition, there are considerable savings in design work and on site operations, because there are no expensive shuttering and reinforcement drawings to be made; estimation on a fixed-price basis is risk-free and the preparation of the estimate itself is cheap.

When steel culverts are used in the design of new road schemes, they can be laid immediately without delaying the start of earth-moving work.

The installation and erection of subsidiary works, such as pipes and outlets prior to earth moving as is usual nowadays, can be avoided by using culverts, as every contractor can than lay and cover light steel pipes without the time for the earthworks being noticeably affected.

The life-span of hot galvanized steel culverts is often estimated to be shorter than that of equivalent concrete culverts, though in many cases steel culverts are more robust because of their flexibility, especially when subsoil settlement is taken into account.

What the client often fails to appreciate is that even a small saving on the purchase price can earn interest over the years, so that at the end of the useful life, a sum sufficient for the eventual replacement of the original would be available.

Calculations on these lines show that a structure with a 60-year life is only 6.2% cheaper than one with a 100-year life.

This facet of the interest earned on savings in the purchase price is however seldom considered by large customers.

The fact remains that the State and also the railways in every country raise loans on the stock market, on which, these days, they are having to pay 6% interest.

Thus, money saved by employing cheaper construction methods means less capital requiring to be raised on the market by the State.

In general, however, this argument is dismissed as being irrelevant as almost all building authorities operate on a strict budget and the predominant view is that all road works should have a useful life of at least 100 years.

Unfortunately, this type of construction has so far found little recognition in Europe, and it is also unlikely that it will, in the near future, be specified in the regulations, as concrete pipes have been over decades.

Neither the German Federal Railways nor the Highway Authorities allow their general use; in fact, this type is permitted by the responsible authorities only in exceptional cases.

On the other hand, this type of construction, which is well suited to the vigorous development of road schemes, is welcomed as a means of price regulation.

For the manufacturer, however, this is still a very unsatisfactory state of affairs, as their sales involve considerable financial sacrifices.

Every inquiry must be assessed technically to determine the plate thickness before a quotation is made.

Apart from the difficulty of carrying out installation over a large area from a central location, it is also necessary to supervise the work during the covering of the culverts.

The technical supervision of the project right up to the final covering is a prerequisite for ensuring the stability of a steel culvert.

It is to be hoped that one day the authorities will accept this type of construction as equivalent to building in concrete, and so promote the cause of this underrated type, that this improvement in building methods will be preserved also for the future.


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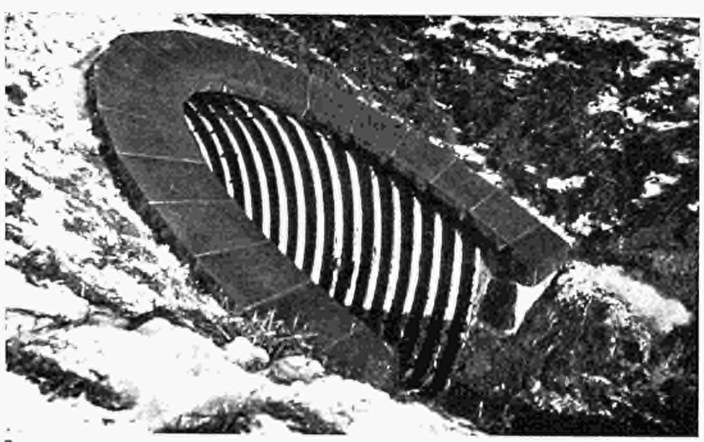
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Siegfried KRUG

The Economic Aspects of Temporay Steel Roads

(Original text: German)

Mr. Thul in his paper [1] told us that "disturbances in the flow of traffic are tantamount to losses to the national economy" (Fig. 1).

I should like to give you an example of the amount of time and money that can be wasted in traffic jams. My remarks concern a motorway blockage, but much the same applies regarding hold-ups in built-up areas, on diversions and so on, and parallel calculations can be made [1,2,3] (1).

The flat steel road at the Butzbach junction on the E4 European motorway was traversed between early April and mid-September — that is, in rather over five months — by a total of five million vehicles. As can be seen from figure 1, the figure for the single month of July, in the peak holiday and travel season, was actually more than one million.

Immediately before the entrance to the flat steel section, the traffic flow was registered on automatic counters. Figure 2 shows diagrammatically a typical traffic load between May 2 and 4, 1964. As you will see, particularly large numbers of vehicles passed on May 3 between 5 and 6 p.m. They negociated the road works without any hold-ups, thanks to the presence of the flat steel section.

The question is, would there have been hold-ups if the steel section had not been there — if the traffic had been channelled, as in previous years, over a single lane with two-way traffic?

By plotting the sum of the hourly flows shown in figure 2 for the reference period as a whole, the curve in figure 3 indicating the total traffic volume is obtained. The peak towards 6 p.m. on May 3 in figure 2 appears as the sudden rise in the curve in figure 3.

Herr Thul in his paper explained that approximately 1,200-1,500 vehicles per hour can pass through a single-lane defile. Taking the upper limit figure, 1,500 vehicles per hour, as the net throughflow for the purposes of the curve in figure 3 it is seen that a blockage is bound to occur when the incoming flow, or afflux, exceeds the outgoing, or efflux. On the graph, the afflux curve is steeper than the efflux curve, indicating jams. The hatched portion of figure 3 gives an idea of the total wasted vehicle-hours, and also indicates the length of the queue and the average waiting time for individual vehicles.

The hatched portion is shown in greater detail and on an enlarged scale in figure 4. Hold-ups begin to develop at the point where the efflux curve A(t) touches the afflux curve Z(t), and cease where the two intersect.

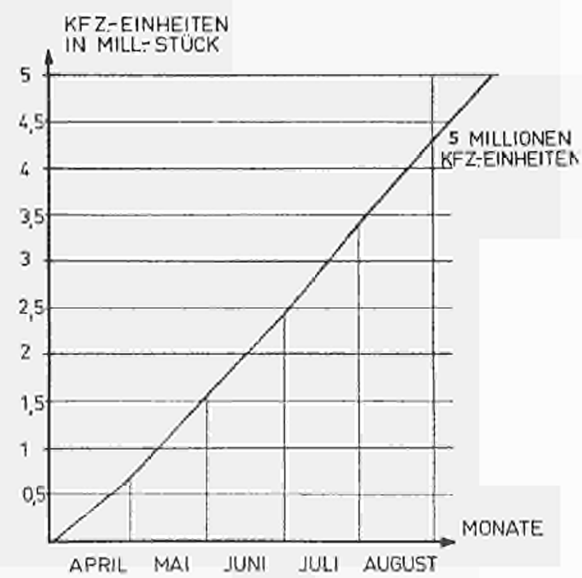

Fig. 1

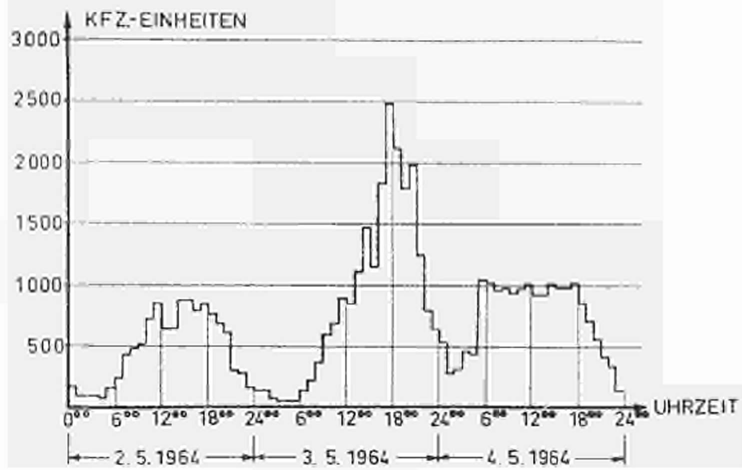

Fig. 2

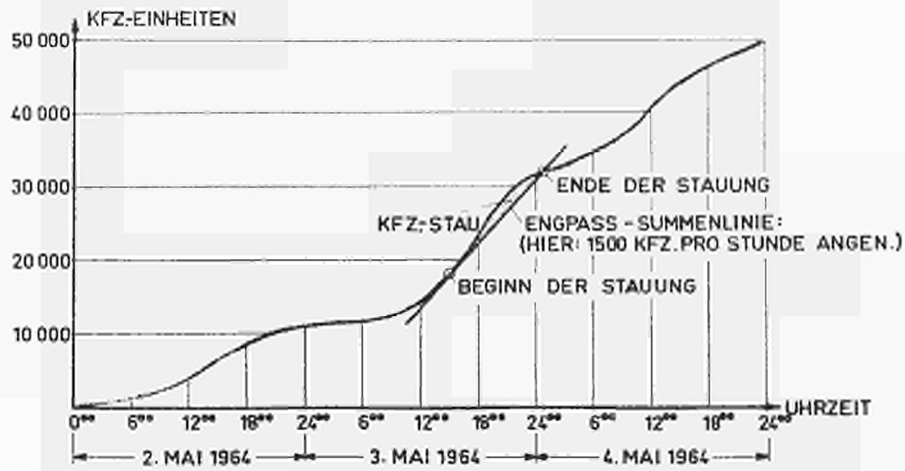

Fig. 3

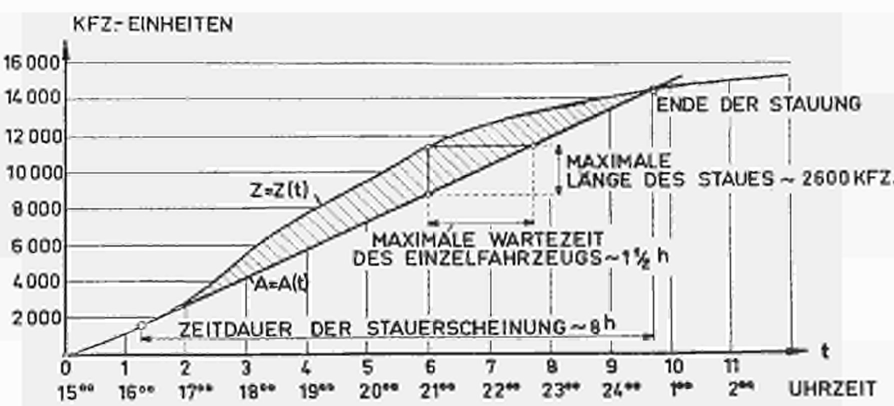

Fig. 4

The area between Z(t) and A(t) represents the total number of vehicule-hours spent waiting (in this instance approximately 14,000); from this can also be calculated the length of the queue and the average waiting time of individual vehicles.

From the figure thus arrived at for the idle vehicle-hours involved the approximate cost may be deduced. Some work has already been done in the United States, and also in Europe on computing time costs on the roads. If we take Jürgensen's figure of DM10 per hour per vehicle, the result in this particular case works out at the very substantial sum of DM140,000. Road users have saved this amount by the use of the flat steel road section.

On other days also densities which, but for the provision for four-lane traffic would have caused delays, were recorded (2).

This example may serve to illustrate the potential of steel from the economic angle also. It is very much to be hoped that full use will be made of the potentialities and advantages of steel construction in connection also with other transport problems. As is clear from the points just made, steel is a modern and economic material.

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3 Reinitzhuber/Semmroth: "Verkehrsprobleme und Stahlbau" ("Traffic problems and Steel Construction"), published in Der Stahlbau, Vol. 2, February 1964.

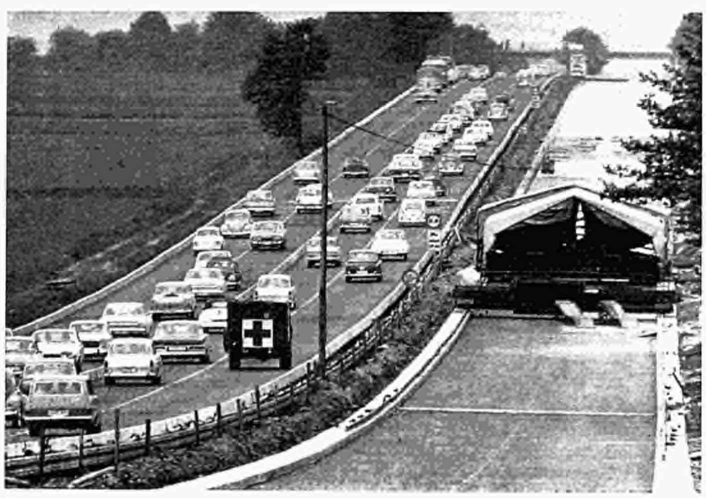
4 Jürgensen, H.: "Zeitkostenersparnisse im Strassenverkehr als neues Wirtschaftlichkeitskriterium für den Strassenbau" ("Time Cost Savings in Road Traffic: A new yardstick of economic operation in road construction"), talk at Hanover Fair, 1963.

Description of photographs

1 — Traffic jam on motorway.

2 — Traffic on flat steel road.

1



2

André-Georges BONNET

Steel Accessories of Motorway Bridges in France

(Original text: French)

In the few minutes at my disposal I can do no more than scratch the surface of this subject. In bridge construction, one must distinguish between bridges which straddle the motorway (over-bridges) and those which carry it (under-bridges).

Over-bridges considerably, improve the appearance of a motorway as thousands of road users see it every day. Under-bridges are built for the latter's comfort and safety.

All these bridges should be provided with balustrading. In addition, under-bridges should be furnished with guard-rails and substantial and comfortable deck joints. For French motorways, which are now being constructed at the rate of 175 km. per year and which involve about 150 bridges per year (100 over-bridges and 50 under-bridges, in round figures), these accessories have been standardized and published in two dossiers (each weighing about 2 kg. and 300 copies of which have already been published) :

— G.C.64 dealing with balustrading, guard-rails and central grilles.

— J.A.D.E.64 dealing particularly with deck-joints of three classes :

  — heavy joints (5 types) for heavy traffic (more than 3,500 vehicles per day), *i.e.* for all bridges carrying motorways;

  — medium joints (5 types) for average traffic (500-3500 vehicles per day), *i.e.* for overbridges for important highways.

  — light joints (5 types) for light traffic (less than 500 vehicles per day), *i.e.* for overbridges for secondary roads.

This is not a negligible market. The balustrading for an over-bridge (long narrow bridge) consumes 5 tons of commercial quality steel, *i.e.* between 10 and 20 per cent of the steel used in the girders or reinforcement in the superstructure, amounting to 5 to 10 per cent of the price of the bridge.

In the same way, a heavy joint in an over-bridge, such as a twin carriageway bridge 30 m. (98ft.) wide, can consume 5 tons of steel and sometimes as much as 10 tons of steel per line of support, which can also amount to 5 to 10 per cent of the cost of the structure.

At the rate of 150 bridges per year, the consumption of steel reaches hundreds of tons.

Furthermore, the balustrading represents much more than 5 per cent of the appearance of an over-bridge. It completes the structure, furbishes it and gives it character. As this expense must be borne in any case, one should at least try to obtain the best possible aesthetic effect from it. The accompanying illustrations (1-5) well demonstrate this.

The deck joint represents much more than 5 per cent of the impression of comfort made by an over-bridge. Our illustrations (6-10) show four particularly comfortable types combining neoprene (noiselessness, shock absorption) with steel (strength, convenience of assembly).

Steel guard-rails used for bridges should be fixed in such a way that they

— do not provide any hard projection,
— do not damage the bridge when they are struck,
— can be easily repaired.

In France, a simple flexible and easily repairable system of fixtures has been developed, in which a square hollow block of neoprene is inserted between a front timber post (which only supports the vertical load of the rail) and a rear steel post (which resists the horizontal blow from the vehicle).

All this standardized equipment (joints, balustrading, guard-rails, grilles) has been developed and publicised in the dossiers produced by the "Service spécial des autoroutes".

This department of the French Ministry of Works prepares the designs and supervises the accelerated construction of motorways in France (240 km in use in December 1962, but 490 in December 1964 and 660 in December 1965).

About 400 bridges were constructed and equipped between 1961 and 1964.

Description of photographs

1 — Urban balustrading, Type S_1, in cold formed sections, 40 kg. per metre. (Motorway A.6 south of Paris).

2 — Rural balustrading, Type S_3, in commercial tubes, 34 kg. per metre. (Motorway A.8 near Cannes).

3 — Rural balustrading, Type S_5, in commercial sheet (20 cm. wide) usually in formed sheet, 45 kg. per metre. (Motorway A.7 south of Lyons).

4 — Suburban balustrading, Type 5_7, in bars, sheet and commercial tubes, 37 kg. per metre. (Motorway A.1 north of Paris).

5 — Bridge balustrading carrying a motorway, Type I_3, in boiler tubes and 16 mm. plates, 30 kg. per metre.

6 — Prototype for a flexible support for steel guard-rails on motorway bridges by means of

— precast concrete supports,
— timber and steel posts,
— square neoprene sleeve between the two.

Note :

— there are no hard projections above the posts supporting the line of rails.
— guard-rails fitted in structures built before 1962.
— simple repairs in case of damage.

7 — Heavy joint, Type A, for motorway bridges (cross-section) a sheet of neoprene is squeezed between the two steel plates

— 6 m. carriageway : 180 kg. of steel per metre
— 8 m. carriageway : 210 kg. of steel per metre
— 10 m. carriageway : 260 kg. of steel per metre

Noiseless, watertight, comfortable. All the components can be replaced.

8 — Heavy joint, Type D.

Prestressed sandwich joint composed of vertical plates separated by tee-shaped neoprene sections, pressed together by articulated spindles under coupels. Consumption of steel : 100 kg. per metre.

Possibility of considerable skew displacements (25 mm. along the joint for 35 mm. of perpendicular play for the structure with a 45 degree skew on which the photo was taken).

Noiseless watertight, comfortable. All the components can be replaced.

9 — Heavy joint, Type E.

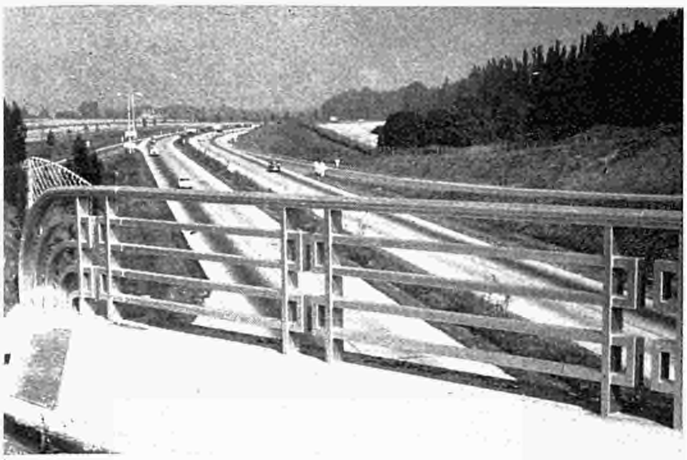
Prestressed joint with a block of neoprene pressed between two spheroidal cast steel supports. Anchorages not bonded to concrete, prestressing by controlled tightening of bolts. (photo taken during placing : the neoprene block will later be lodged between the two sections).

Steel consumption : 80 kg. per metre.

Noiseless, watertight, comfortable. All the components can be replaced.

10 — Heavy joint, Type G.

Steel and neoprene comb joint. Play possible : 5 cm.
Consumption of steel : 80 kg. per metre.
Comfort and noiselessness : neoprene.
Rugosity : quartz sand bonded with epoxy resins.
Replacement of the joint on an old structure.
Sealing with epoxy resins.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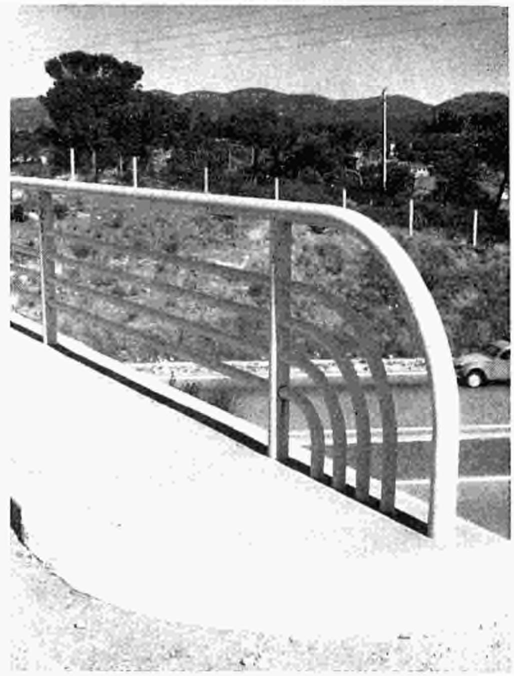
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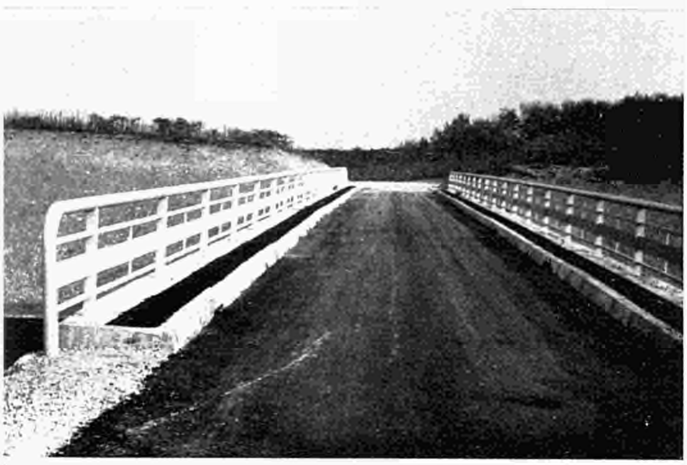
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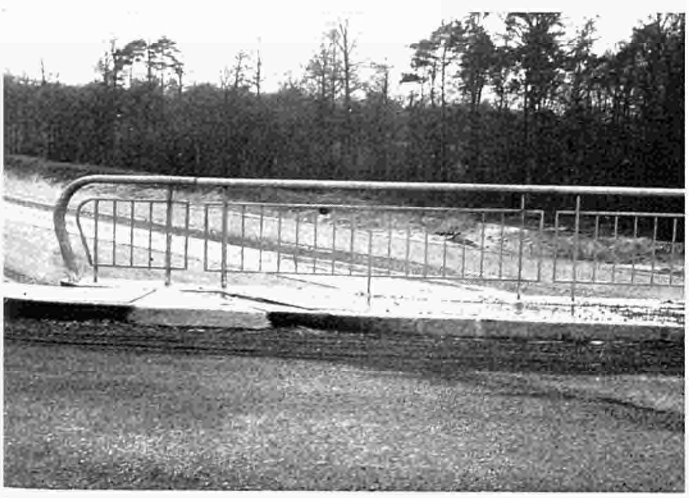
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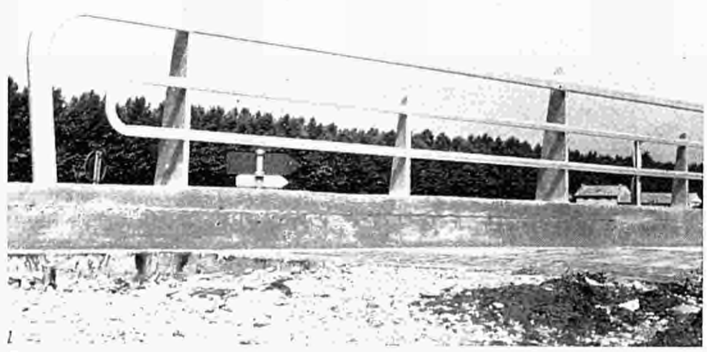
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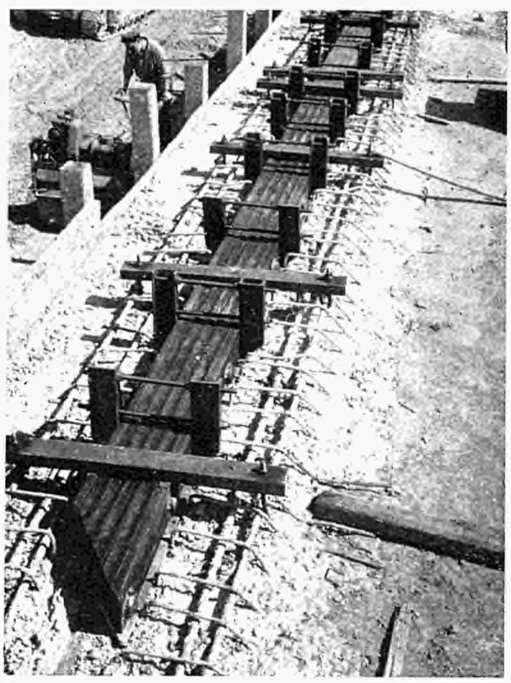

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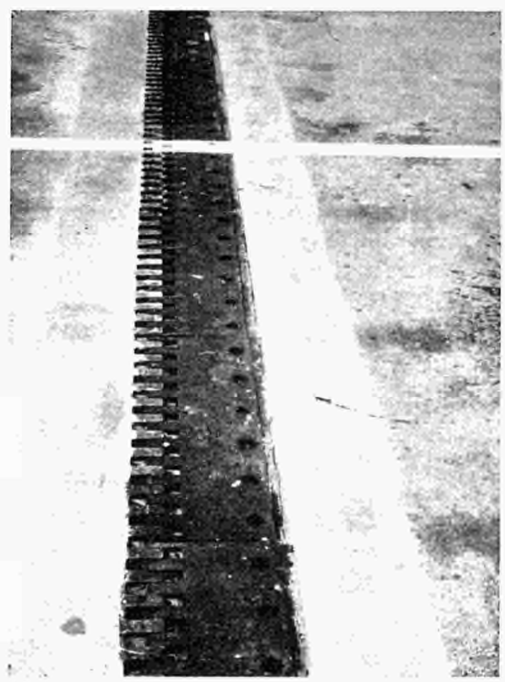


10

Patrice DECAIX

The Evolution of Street Furniture in France

(Original text: French)

Preamble

As there was already a good highway system in existence, France concentrated during the post-war years on building up its vehicle park, which soon became the largest in Europe. This economic and political choice was not the same as that made in other neighbouring countries which preferred to build roads. This is the reason for the comparative position of European motorways in 1964, there being some 400 km. in France but many more in adjoining countries, as for instance the 3000 km. in existence in Germany.

These preliminary remarks will help to explain the present situation in France with regard to street furniture and the part which has been reserved for the use of steel in this sector. The existing equipment is traditional in conception and sometimes of foreign design because the size of the market does not justify special investigations or research into equipment.

Equipment

From the technical aspect let us examine briefly the situation with regard to equipment. This can be divided into two classes :

— Accessories proper: fences, anti-dazzle screens, lamp standards, guard-rails, gantries, traffic signals, balustrading and footbridges.
— Ancillary structures: small bridges, culverts ...

Accessories proper

Fences are of traditional type, in barbed wire; simple, corrugated or interwoven mesh, less frequently in trellis or welded mesh; protection is normally afforded by galvanizing and the use of either bituminous or zinc chromate paint.

A rough breakdown shows that there are several principal types, according to the function envisaged : fences for cattle, small and large game; very strong fences for use in towns. These types are defined by their height (1.20 to 2.50 m.) the mesh (50 × 50 mm. being the most usual) and the diameter of wire (normally at least 3 mm.). The posts are either angles or tee-sections (80 × 80 mm. for strutted straining posts and 50 × 50 mm. or 40 × 45 mm. for intermediate posts).

Very strong fences of expanded metal are made in panels 1.20, 2 and 2.40 m. in height and 2.40 m. in length.

Anti-dazzle screens, planned but not yet installed in steel, are in expanded metal (panels 1.50 m. high and 4 m. long in 15/10 mm. sheet.

Lamp standards, the use of which for public lighting is rapidly increasing, are based on the models designed for motorways. Of purely French origin, such as the tubular standard designed in 1950 for the Autoroute de L'Ouest, "fashion" has evolved their shape towards an upright or leaning standard with a simple or double bracket, conical or pyramidal in form and generally octagonal or circular in section.

From the fabrication point of view there are three types : folded sheet (for motorways), tubes and folded sheet with tubular brackets. Usually fabricated in weldable mild steel of quality A.42 with a thickness of about 4 mm. for present-day models, the standards are sometimes made in medium tensile steel with adornment, such as shafts with short lengths of conical sleeve of varying thickness (1 to 2.25 mm.).

For motorways, the column is attached to standardized sole plates (300 × 300 mm. and 400 × 400 mm.). The usual height is 12 m. with a 3 m. bracket or 10 m. with a 2.50 m. bracket. Brackets of 4 m. or heights of 15 to 20 m. are met in public lighting, such as the 20 m. standards at the junction of the Orly motorway with Route Nationale 7.

Protection against corrosion is achieved by painting, by galvanizing or zinc spraying, metallization to a thickness of 80 microns together with a coat of zinc-rich paint being recommended to reduce maintenance costs, which have been estimated to amount to 1.2 per cent per year of the erected standard. The use of stainless steel base-plates seems to give appreciable results for the protection of the foot of the standard-column.

Two types of **semi-flexible** guard-rails with profiles A and B were standardized in 1964 by the Highways Dept. of the Ministry of Works (Fig. 1 and 2). They are in mild steel of quality A.37, and have a thickness of 3 mm. Type A is based on a model already in use, but with slightly larger sections and moments of inertia, the weight per metre being 12.8 kg. Type B is based on another model with a slightly smaller moment of inertia and weighing 12 kg. per metre. Protection by galvanization has been successful.

The supports, which are not yet standardized, are either channels 60 × 140 × 60 mm. for type A or channels. 70 × 150 × 70 × 6 or 80 × 120 × 80 × 5 or two H sections 125 × 125 for type B, the height above the ground being less than 70 cm., the anchorage being 1 m. and the distance between supports usually 4 m.

Flexible guards composed of cables have so far only been installed over one length of 150 m. One item to note is the patented cable-fixing device which rests on a notch in the support and which can slide only under the effect of a blow.

Sign gantries, which are not yet very numerous because of the small mileage of motorways, can be classified, according to their fabrication, as being of heavy or light tubing or tubular lattice construction. The signs may be of panels lit up on the outside or of boxes with interior lighting, the latter solution being preferable from the point of view of lighting technique.

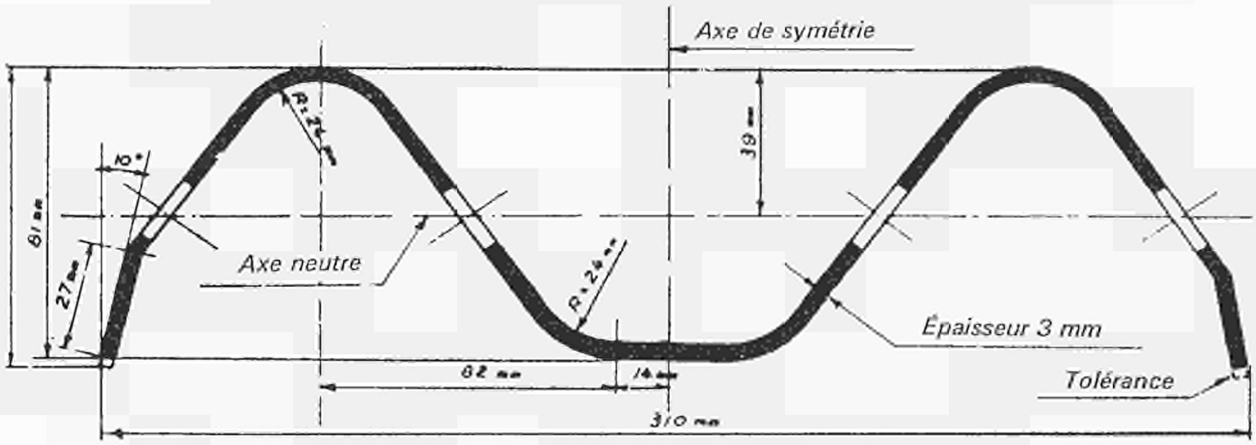
Gantries are classified into 9 types, with spans of 10, 14 or 18 m. or more, with sign-boards with areas of 10, 14 or 18 sq.m. or more.

A gantry in tubular steel (e.g. 323.7 mm. o.d. × 7 mm. thick or 355.6 mm. o.d. × 8 mm. thick) can be used for any project.

PROFILE A

Overlapping joints (covering ⩾ 30 cm.)

Steel used: quality A 37 according to A.F.N.O.R. norms (minimal quality)
Thickness: 3 mm. (tolerance on thickness as per steel-mills regulation for hot-rolled hoop and strip)
Length: useful length between posts - cross-section: 400 cm



DISPOSITIONS GÉNÉRALES DU JOINT

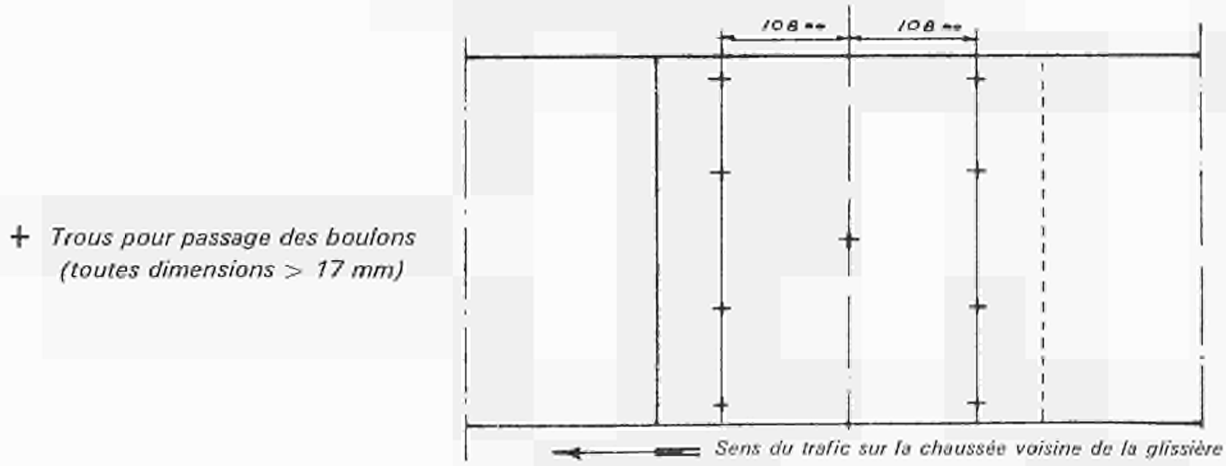


Fig. 1

PROFILE B

Fish-plate joints

Steel used: quality A 37 according to A.F.N.O.R. norms (minimal quality)
Thickness: 3 mm. tolerance on thickness as per steel-mills regulation for hot-rolled hoop and strip.

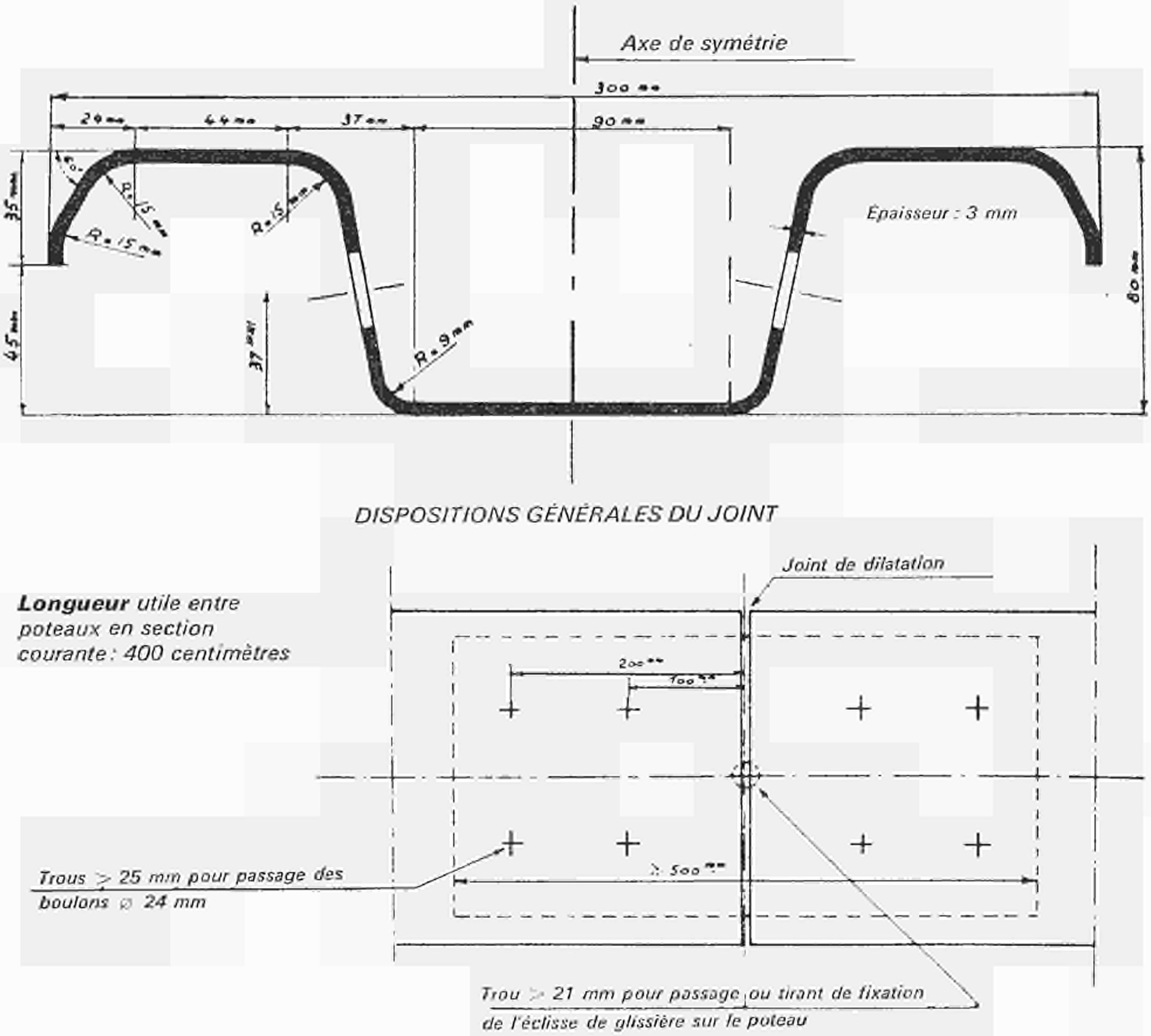


Fig. 2

Erection is quite easy, the weight varying from 1 to 3 tons for the existing types and the span reaching 23 m.

A number of examples of heavy gantries in tubular lattice construction have been erected recently. Their geometrical shape follows the bending moment diagram. The span is about 17 m., the sign boarding, which is provided by two lighted boxes, having an area of more than 30 sq.m., while the weight is about 6 tons.

A light tubular lattice gantry, as well as one in cold-formed sections, is being designed.

Footbridges, by their technical concept, cost and dimensions, come very close to a bridge structure. A classical example may be cited : a portal-frame footbridge in mild steel, of quality A.37, comprising two spans of 14 m. with headroom of 5.15 m., with expanded-metal balustrading and two lateral approach stairway ramps, the total width being 33.50 m. Other examples are the tubular-steel footbridge, 70 m. in span, constructed over the Argens in Var and a footbridge at Orly Airport which has already been described in another paper.

Balustrading has been specified by the Highways Department. Classified as balustrading for over- or under-bridges, for urban or rural areas, they comprise the following types: horizontal members (with tubular rails), multitubular with rails on both sides or tubular, with panels, with wide sides, with square mesh concertina sides, or vertical. The balustrading with wide sides can be made with stainless steel expanded metal.

Ancillary structures

Steel segments, the characteristics of which are international, are used for small bridges.

Numerous examples, in particular on the Vienne-Valence motorway, could be cited where rapid installation and simple replacement are the qualities which make their use preferable to that of concrete.

The same is true also of box culverts and corrugated culverts for drainage.

The equipment for temporary roads, or similar material, has had little practical application, but it represents a sector worthy of our attention when considering the wider use of steel.

Development of fittings

The newly-issued French list of street furniture shows a certain character which seems to have been released from traditional lines by several factors, as follows:

- the development of the construction of motorways which can normally be estimated at 175 km. per year, but may be increased by new financial measures.
- the intensification of safety measures on account of the increase in traffic,
- aesthetic research which, for certain materials, seems to be more intensive in France, on the part of the authorities.
- the availability on the market of other competitive materials.

What, therefore, is the possible development of steel to meet these different factors? It is governed by two aspects of the problem.

- the first is technical: the design of new equipment with an aesthetic character and an improvement in the method of protection, so that maintenance costs are kept down.
- the second is economic: the place to be given to street furniture in the general market for steel.

Let us first examine the *present solutions from the technical point of view.*

The principal essentials are as follows:

- strength and elasticity, according to the function desired:
- weight, in order to permit rapid erection or replacement; with the minimum of handling equipment;
- protection, to increase the durability and reduce the cost of maintenance;
- aesthetic.

Fencing

Steel is the only metal which complies with the requirements demanded for fences of great height; efficient galvanizing, conforming with standards, ensures that they are properly protected.

The use of plastic-coated netting with a web of about 2 mm. galvanized wire is at present being tested for fences of medium height upwards. Several examples have shown that the aesthetic problem is solved when the colours are matched with those on the site.

The use of stainless steel netting must be confined to particular cases.

Lighting standards

The mechanical characteristics of steel lend themselves to the design of tall modern shapes to meet the demands of the latest lighting techniques. A standard formed from folded plate, of hexagonal section and 15 m. high, fitted with a double cross arm, 4 m, long, will soon be erected on the motorway north of Paris. Similarly, a standard with short brackets for 2, 3 or 4 lamps, with heights of 12, 14 and 16 m. following another technique, ensures the lighting of large areas.

It is useful in the case of lamp standards, as in the case of guard-rails, to draw attention to the undeniable superiority of steel over other metals from the point of view of safety. At the time of impact the energy stored up is much less in the elastic zone than in the case of another metal with *equal* yield stress but lower modulus of elasticity, but it is *much greater* in the plastic zone by reason of the value of the resistance to shear or fracture. This has been confirmed with a standard in mild steel of quality A 42. In an accident the standard buckles, but does not break.

In regard to protection, a metallic coating (50-110 microns thick produced in particular by zinc metallisation) reduces the cost of subsequent painting. An efficient form of protection can also be provided by fabricating the standards from galvanized steel plate; tests are being made.

At present the use of stainless steel is confined to the bases of standards. It is also envisaged for the fabrication of standards at a price which is higher but still competitive, bearing in mind the superiority of the mechanical characteristics in the production of modern shapes.

Guard-rails

We are awaiting the results of tests organized by the Ministry of Works and also those on concrete rails at the Highways Department's testing station. Further tests on steel equipment, already standardized or under examination will allow us to lay down the dimensions and spacing of the posts, in particular.

For the equipment of bridge decks or the protection of structures, the Highways Department has a special specification which comprises a double support with posts of timber or steel sections, with a square neoprene coupling inserted at the level of the rails, fixed in a precast concrete socket or to a removable base plate. A variation of this equipment with a timber post and a steel section, and two rails, can be cited as an example. This has been used on the bridge over the River Giors which crosses the valley at a height of 40 m. In case of accident, protection is ensured both by the guard-rails and by expanded metal balustrading.

Gantries, brackets and direction signs

New regulations being prepared for the sign-posting of motorways will allow the fundamentals of the problem to be better established and will facilitate research towards a technical solution for signs and supports, by means of modular panels or some other type adapted for easy installation or replacement. Here too galvanized steel sheet can provide an economic solution.

Technically steel is the only material which permits the production at reasonable cost of large-span gantries and large areas of signs. Some have already been erected; others which are being studied, offer an undeniable aesthetic character, associated with lighting techniques in conformity with the requirements of the authorities.

The technical aspect of the problem having been briefly described, it only remains to take a look at the problem from the economic and production aspect.

It is difficult to assess with reasonable accuracy of the tonnage of steel involved in the street-furniture market because, for certain fittings, such as sign-posts, rules are at present being formulated by the Service Spécial des Autoroutes, while for others, such as structural accessories, estimates are subject to situations depending on place or time (in particular, economic requirements which necessitate the use of temporary steel highways).

It should also be borne in mind that the volume of work undertaken depends on the funds (often fluctuating) made available to the appropriate authorities.

Be that as it may, in the light of statistics * or by hypothetical deductions from the regulations or the financial possibilities, one can establish an amount, in so far as highway fittings proper are concerned, of the order of 50,000 tons for a period of 15 years, or about 3,500 tons per year of which guard-rails would account for 50 per cent.

This estimate represents only a percentage of the overall market for street furniture. It is also related to the installation of a fitting and must be increased by a factor if allowance has to be made for eventual replacement (as in the case of guard-rails and lamp standards).

Conclusion

Although the market for street furniture may seem to be small compared with that for other equipment, it is of considerable value if the psychological effect produced on millions of road users by the direct and visual acquaintance with a particular use of steel is taken into account. It can be concluded, therefore, that in France as well as abroad, the effort needed to make improvements or to design new fittings can only be beneficial in the promotion of the general use of steel.

* Take the case of guard-rails, in particular. In the autumn of 1962, for 240 km. of motorway, the length of guard-rails was 71 km., 54 of which were in steel. On all motorways and trunk roads there were 171 km., 85 being in steel. One can therefore allow for the annual installation of 45 km. of guard-rails per 100 km. of motorway and about 40 km. for trunk roads.

Description of photographs

1 — Lamp standard of seamless drawn steel tube, Autoroute de l'Ouest.

2 — Lamp standard of folded sheet, Autoroute Sud de Paris (Orly bypass).

3 — 20 m. lamp standard of folded sheet for road crossings.

4 — Lamp standard of folded sheet with 4 m. bracket, Strasbourg.

5 — Lamp standard of folded sheet with 4 m. bracket, Lyons.

6 - 7 — Types of lamp standards for highway lighting.

8 — 10 m. lamp standard of corrugated steel sheet, star-shaped in section with four crossed cast-iron brackets, Place de l'Etoile, Paris.

9 — Incandescent iodine lighting, Place de l'Etoile : 24 standards each with four 1000 W. lamps.

10 — Type of 10-16 m. lamp standard with short bracket for lighting of large spaces.

11 — Type of 15 m. lamp standard of seamless drawn tube.

12 — Balustrading of bridge at Sèvres.

13 — Guard-rail supports, type B.

14 — Flexible guards composed of cables (design)

15 — Conventional bracket-type road sign, Autoroute Esterel — Côte d'Azur.

16 — Tubular-steel gantry (assembly) with a span of 10-20 m.

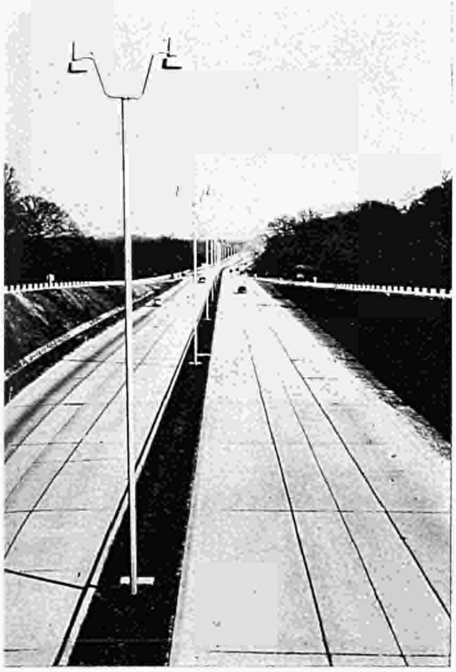
17 — Tubular-steel gantry (for road signs) with supports for sign boarding.

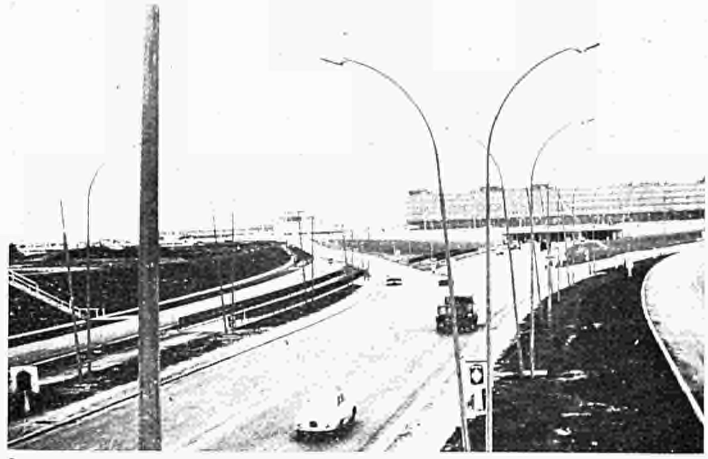
18 — Heavy gantry in tubular lattice construction, span 17 m., lighted boxes; sign boarding area 30 sq.m.

19 — Light tubular lattice gantry and bracket road sign (under study).

20 — Folded-sheet gantry and bracket road sign (under study), span 10-14 m.

21 — Tubular-steel footbridge over R. Argens, Department of Var, span 70 m.





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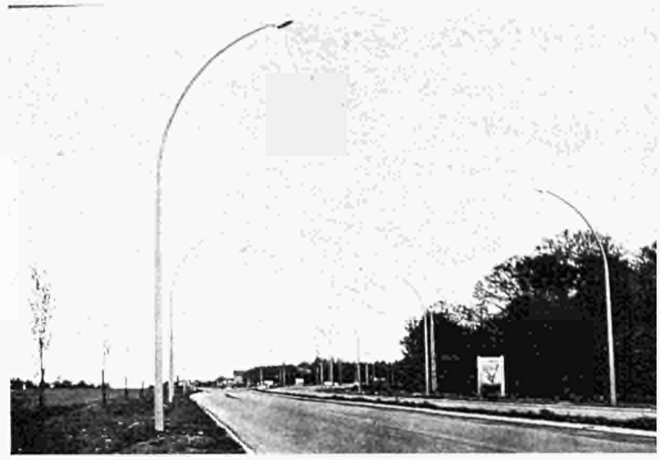
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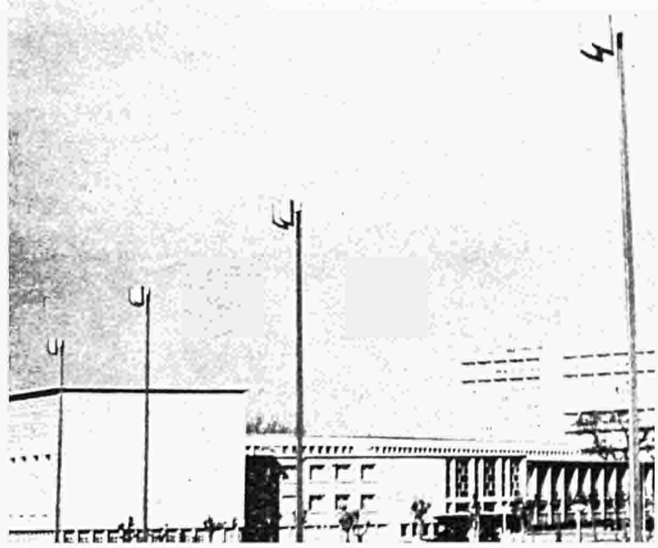
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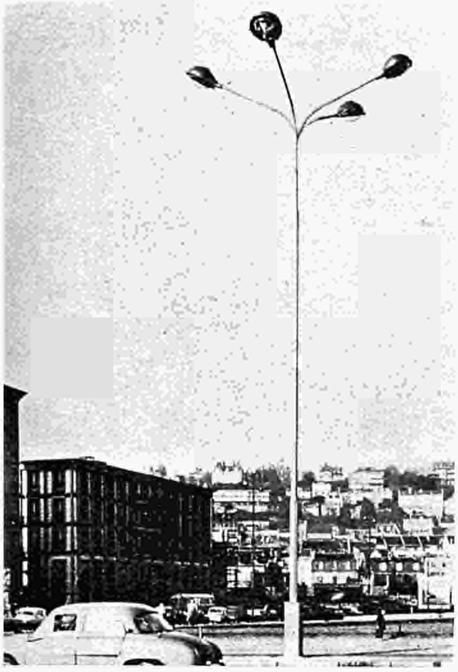

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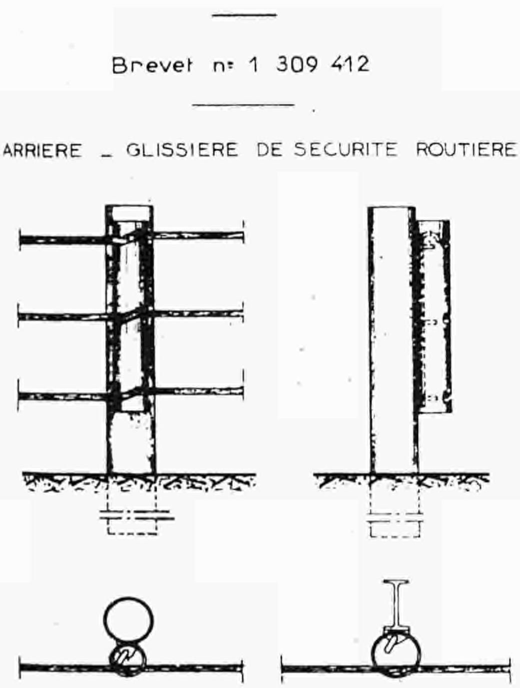
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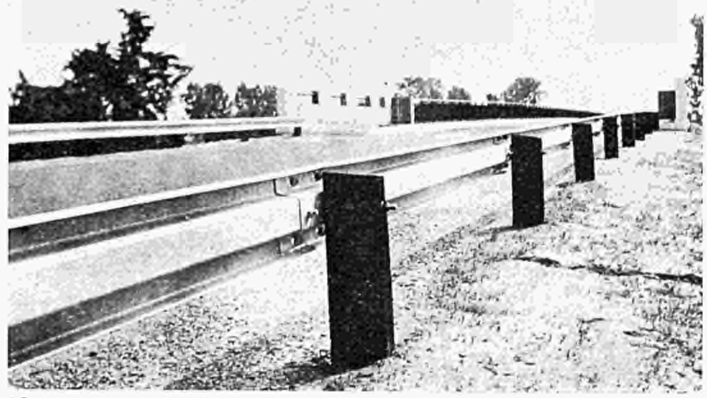

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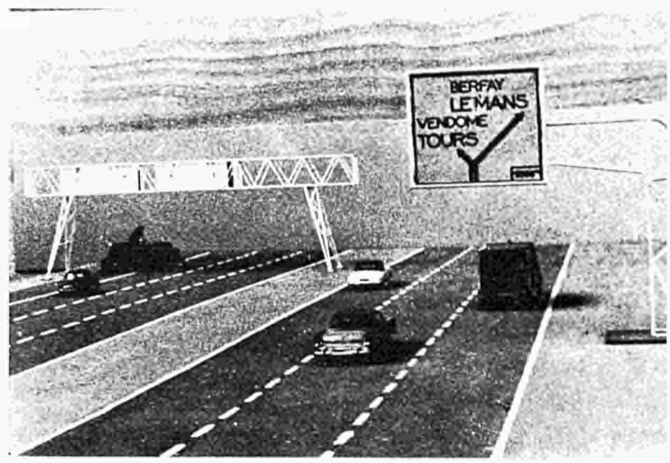

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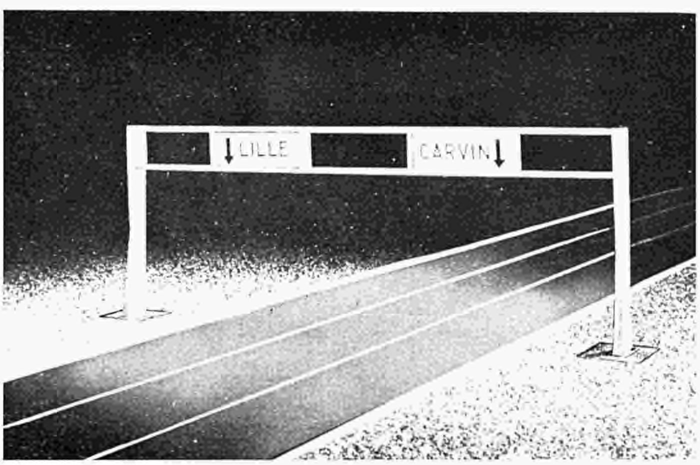

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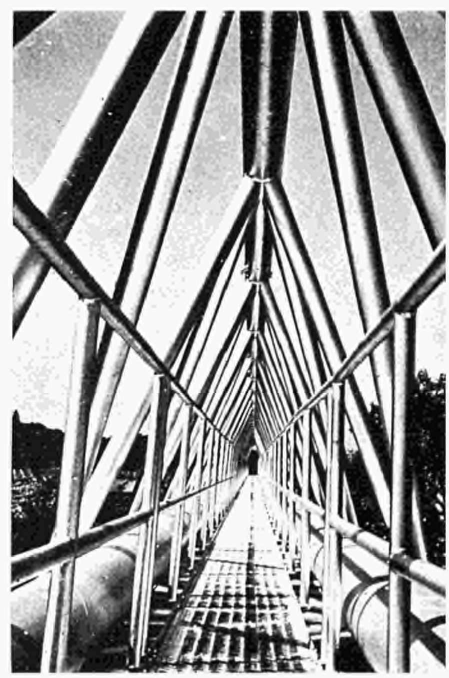
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Nino SANSONE

An Efficient System of International Road Signs as a Safety Measure and a Means of Improving Traffic Flow

(Original text: Italian)

My concern is with an efficient system of international road signs as a safety measure and a means of improving the flow of traffic. It has been asserted that nowadays a road plan cannot be regarded as complete unless it provides in clear and precise terms for an adequate system of regulatory and informative signs. Consequently, an efficient system of signs comes to mean something which started off as one of many roadway accessories and has now become an integral part of the road itself. My aim is to emphasize the difference between a mere collection of signs of various kinds strung out along the road and an efficient system of signposting which has been studied from a technical point of view and the component items of which are both uniform and functional throughout a road system over a very large area. No one will deny that the problem has now assumed scientific dimensions, but there are still a great many road authorities who fail to put into practice the praiseworthy convictions expressed in theory, or in official speeches by their members.

Results can only be achieved by entrusting specialized engineers with the task of preparing a general plan for detailed signposting. Where it is evident that the ensuing advantages result in increased safety and in freer-flowing traffic there should be no further procrastination in solving this major problem.

Leaving on one side the classification of signs into vertical and carriageway, or regulatory and informative, it would clearly be unforgivable to refer to vertical informative signs at this Congress without at least mentioning a few of the extremely important applications of steel in this field, particularly as regards the future. European road legislation generally requires vertical sign panels and their supports to be of metal or concrete, though timber may also be used. Prevailing usage tends to metal supports, which can either be U-section, triangular or tubular. Now that internally lit signs are giving way more and more to panels with reflecting surfaces, the use of metal signs and supports is bound to predominate over all other types because of ease of transport, installation and, particularly, maintenance together with increased length of life.

Amongst roadway accessories used to promote safety or traffic flow, special mention should also be made of foglights used on main roads and on stretches of highway where fog is recurrent of anti-dazzle fences on central reservations, protective works, rock and avalanche barriers, of which we have seen some extremely interesting examples. Such a system also covers stripes, carriageway markings and boundary lines which are still on the pavement in some countries. Mud and snow have been and continue to be a nuisance, in that they cover up boundary markings, making it impossible for drivers to find the nearside. At night this is of great importance as regards safety and traffic speed. An important development in this respect, in view of its successful results, has been the use of vertical metal posts 2-3 feet high which the driver can sight a long way ahead. These indicate the alignment and direction of the road up to 600-1000 yards and also have the great advantage of inducing the driver to keep in to the side of the road as much as possible and away from the centre to which he tends to veer.

The standardization of both highway codes and road signs has been studied by technical experts and politicians for many years at O.T.A. (London), E.C.E. (Geneva) and E.C.M.T. (Paris). Concrete progress has already been made towards standardizing codes, at least as regards drivers' behaviour, but little has been achieved as regards signs because many points of view still differ widely. The question of the international standardization of road signs is of great importance on account of the improvement in road safety and traffic flow it would promote. I do not claim to be original when I say that the ultimate aim of a functional system of road signs is to enable a motorist using a road for the first time, maybe at night; to feel just as much at ease as if he were perfectly acquainted with its characteristics and shape. No one can deny the importance of this objective, which will be seriously compromised if the hoped for progress in standardizing road signs is further delayed.

I do not propose to go into the question of road sign costs and their productivity from the time saving aspect. Mr. Krug has told us how much an hour's use of the road costs for a car. A figure of 175 Belgium francs was mentioned. Owing to lack of time I must pass over all these points, and so, in conclusion, I would say that what road traffic needs from this Steel Congress is an enormous quantity of steel for a great variety of purposes. It is needed for structures such as bridges and elevated roads; it is required for movable roads both at ground level and overhead, (the importance of which has been so ably demonstrated by the *rapporteurs* to this Working Party): it is required for vertical parking schemes, which have been dealt with by another working Party; it needs it for protective works on bends, for traffic islands and for vertical informatory signs, which must be efficient, functional and internationally standardized. All this is a duty owed to the motorist. Improvements must relate not only to main roads and motorways but to all other traffic arteries, even if only of local importance. In the interest of increased speed and freedom from congestion we must give effect to our wishes of ensuring both material and social productivity.

Robert H. WHITE

Utilization Maintenance of Lamp-Posts-of Stainless Steel

(Original text: English)

Dr. Schultheis Brandi has drawn attention to the important role of steel in road lighting and safety devices. It was also interesting to note examples of lamp-posts clad with stainless steel sheet over the lower part of the lighting column.

Throughout the world our highways and byways are expanding as transportation and communications increase, and with this expansion comes the widespread demand for better lighting to increase the safety on these roads. Consequently, the number of lamp-posts which is annually required is growing rapidly, with an attendant expanding maintenance problem. The constructional material of these posts is diverse, some claiming freedom from maintenance. Therefore, if steel is to offer itself as a competitive material, attention must be paid to maintenance and a labour saving advantage proven.

With this problem in mind, and to combat the growing cost of maintenance. The International Nickel Company developed and is actively promoting the stainless steel lighting column. Using the superior mechanical and physical properties of nickel chromium stainless steel a lighting column can be designed with a low wall thickness which not only produces a light weight lamp-post, but is also conducive to road safety.

The use of the correct grades of stainless steel gives complete freedom from atmospheric and soil corrosion and therefore treatment of buried posts or costly painting of the column at regular intervals is abolished. It must of course be remembered that the labour required for maintenance can be both scarce and costly and the situation will normally be more severe in succeeding decades.

Stainless steel lamp-posts are already in use in the U.S.A. and Great Britain. Photograph 1 shows a Type 301 stainless steel lamp-post in Philadelphia, Pennsylvania, U.S.A. Mounting height is 30 ft. (9.15 m.) and wall thickness is 0.072 in. (1.83 mm.). A post of similar dimensions is shown in photograph 2 and this installation is in St. Paul, Minneapolis, U.S.A. A Type 316 stainless steel 15 ft. (4.57 m.) high lamp-post is shown in photograph 3. This is a British Class 'B' lamp-post suitable for residential and side road lighting. Wall thickness in this case is 0.064 in. (1.63 mm.) and diameter of the upper column is 3 in. (7.62 cm.). Photograph 4 shows an 11 ft. (3.37 m.) high 3 in. diameter (7.62 cm.) Type 321 stainless steel lamp-post erected in Edinburgh, Scotland, by the Edinburgh Corporation Transport Department. Since the installation of the first lamp-post in 1954, a total of 919 have been erected and services has been excellent, no maintenance being required, apart from occasional washing. to remove dirt.

The nickel-chromium stainless steel lamp-post development offers a challenge to manufacturers in both design and fabrication techniques if the excellent properties of the material are fully utilized. As lighting authorities throughout the world realize the economic benefits accruing from the installation of such lamp-posts so our future streets will not only be lit by maintenance-free posts, helping the rate or tax payer, but will also by their aesthetic appearance add prestige to our streets and highways.

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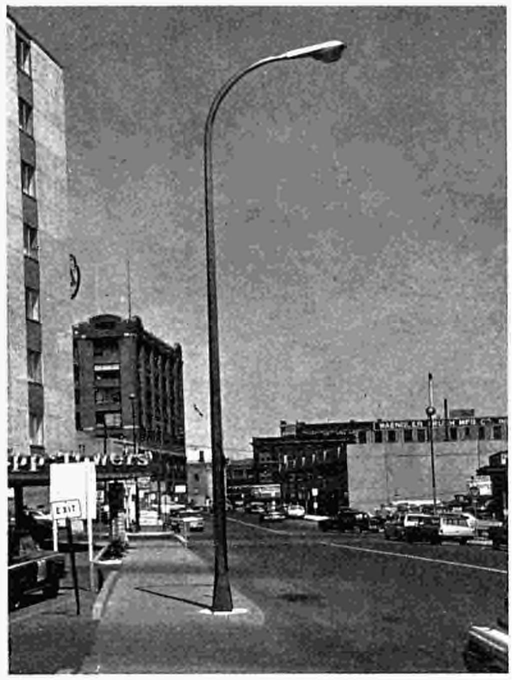

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B. DE BORDE

Special Steels and Street Furniture

(Original text: French)

The ever-increasing traffic in our towns and on our roads demands the creation or revision of ancillary equipment. In passing, it would be wrong to look upon such items as mere accessories, because they are essential from the point of view of both safety and service.

Steel is normally the basic material for street furniture which must combine the highest standards of durability with minimum maintenance costs and safety.

For some of this equipment the claims of weldable high strength alloy steels must not be ignored as, generally, they prove economic in service because they allow a reduction of section and weight in addition to providing a far better resistance to corrosion, with or without surface treatment, than other steels. Such a steel, the Corten, is being used increasingly in America for welded structures, chassis of vehicles, bodies and frames of mineral wagons.

But doesn't it ever occur to anyone that stainless steel, with all its attractive qualities, could make a valuable contribution to the equipment in our streets, or on our highways or motorways, for example, as lamp standards, balustrading, expanded-metal anti-dazzle screens, footbridges or direction gantries?

In spite of its high initial cost, stainless steel has proved to be economic, and therefore competitive in applications other than those in the chemical industry, for example, in the building industry for window frames, curtain walls and other prefabricated elements where ticknesses of the order of 1 mm. have been used.

In fact, stainless steel is :

— a durable material unaffected by the weather,
— a weldable material possessing such qualities as high tensile strength, shear resistance, resilience and endurance to fatigue.

In street furniture stainless steel, the new and modern material, will ensure :

— greater safety on our highways,
— an appreciable saving in weight (lighting standards weighing 1 cwt. instead of 4.)
— the elimination of the costs of maintenance and periodical painting,
— the elimination of the need for long-term replacements.

In addition, it will resist, better than any other material, the action of salt or other corrosive agents occasionally thrown on our roads.

Because of all these factors, experience and calculations show that it is possible to amortize the value of certain types of equipment in stainless steel in less than fifteen years.

This is why rapid developments are taking place in stainless steel lighting standards in the U.S.A. My friend R. White of London will talk to you about them. In Europe, the conditions for such a development appear to be satisfactory as the inherent properties of the material allow thicknesses as little as 1 to 2 mm with an excellent factor of safety.

I will end by describing another item, the French applications of which appear to be first of their kind anywhere in the world : it consists of stainless steel balustrading for bridges. It has been used in the Wilson Bridge at Saint Denis, near Paris, and the new bridge at Choisy-le-Roi which crosses the River Seine and the Paris to Lyons railway track.

Photographs 1, 2 & 3 show the elegant, but unusual and, if I may say so, frail appearance of such balustrading, being erected in prefabricated components for the Wilson Bridge at Saint Denis.

The engineer has in fact designed and fabricated a girder of great strength. The web consists of triangulated bars, 10-12 mm. in diameter, with rakers to resist impact, the bars being assembled on two chords.

The lower chord, which is welded to stirrups which are buried in the slab, consists of plate 6-10 mm. thick and 250-300 mm. wide. The upper chord, which is lighter, comprises two strips, about 2.5 - 3 mm. thick, which have been folded in a press thus providing very great rigidity. The bars are welded to the chords and, moreover, cross the thick lower chord. It should be stated that the welded zones have the same high strength as the adjoining metal, *i.e.* 70 kg, per sq.mm. ultimate load and 55 kg. per sq.mm. shear strength in the usual normalized condition, and 100-140 kg. per sq. mm. ultimate load and more than 65 kg. per sq.mm. shear strength in the commercial state.

This stainless steel balustrading, which has been patented, will withstand the most severe tests and, thus, the worst operating conditions that might be imposed. The shape can be changed to suit any architectural requirement.

Description of photographs

1 - 2 — Views of the stainless steel balustrading from the left and right hand sides of the Wilson Bridge looking towards Saint Denis.

3 — Details of the balustrading for the Wilson Bridge, showing the triangular arrangement of the bars.


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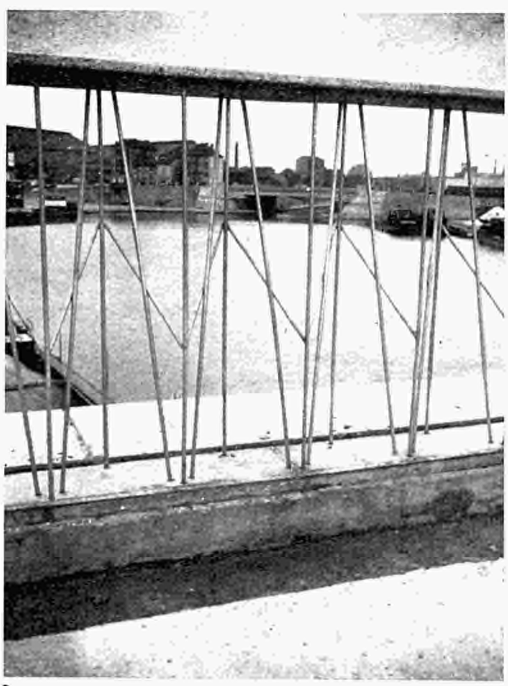
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Raymond PELTIER

The Use of Steel in the Building of Roads in France

(Original text: French)

Since 1961 radical changes have taken place in the methods used in France of building concrete roads: whereas up till then they had been much the same as anywhere else in Europe, this is no longer true today. In point of fact, round about 1960 French civil engineers found themselves obliged to adapt and alter the traditional techniques.

The object of the changes was, firstly, to strengthen this type of road in order to enable it to stand up to the increasingly heavy volume and weight of the traffic of today; secondly, to improve the surface, which directly affects the road user's comfort and safety, and lastly to reduce the cost, since it was found that in the competition between concrete and asphalt roads the latter were drawing ahead.

The French designers had the choice of two apparently equal alternative:

(a) to incorporate steel in roadmaking concrete in such a way as to produce not actual reinforced concrete in the usual sense of the term, but a kind of braced or strapped concrete in which the purpose of the steel was not to prevent fissuring altogether, but to prevent the fissures from opening. This would have required from five to ten kilogrammes of steel per square metre of roadway, depending on the type of terrain and the volume of traffic;

(b) to make the concrete easier to handle, so as to make a first-class surface, and to aid the contractor by pushing up his daily output, thus enabling him to keep production costs down.

The second alternative was eventually chosen, and, thanks to the manufacture in the United States of powerful "slip form" machines, French roadmakers are now able to construct a kilometre of motorway a day. The results have been well up to expectations: the excellent surface produced by this equipment is greatly appreciated by road users, who often have only this to go on in judging the quality of a road. Moreover, production costs have remained competitive for major highways, despite the extra thickness of the concrete.

Consequently, France has entirely discontinued the use of steel bracing or strapping for concrete roadways. This Congress has therefore come too late from this point of view, as there would no longer appear to be any possibility of securing a change in the policy of the French Ministry of Public Works. At the same time, I am convinced that active representations four or five years ago by those concerned with increasing the steel utilization might well have led to a similar concentration on alternative (a).

Although steel-less roadmaking seems definitely established in France for several years to come, it is quite possible that other and both technically and economically more satisfactory methods will be adopted at some time in the future. And since in roadmaking, as in every other field nowadays, the techniques of the future have first of all to be studied at laboratory level, I may add that our *Laboratoire Central des Ponts et Chaussées* has for several years been working on a new technique of French origin, using prestressed concrete. Of course we are still only at the experimental stage, but some extremely encouraging results have already been obtained. If adopted, these new methods would mean the use of substantial quantities of high-resistance special steels.

My conclusion then, is that while steel has lost a battle on the roadmaking front in France, it may well be that a major research drive, supported by the iron and steel industry, would enable the lost ground to be regained, provided the prestressed concrete method could be successfully developed.

Ivo POTENZA

Temporary Flyovers
The "Mobilpass" Overbridge and Road and Railway Footbridges

(Original text: Italian)

Introduction

For some time now many organisations, firms and engineers, anxious to help in solving the complex problems of modern traffic, have been paying close attention to new developments.

Structures such as the "Mobilpass" overbridge, road and railway footbridges have recently been designed and introduced with great success as a means of dealing with urgent problems at various types of junctions and crossings.

The solutions I propose to describe represent some of the innumerable applications of steel and show how irreplaceable it is for certain purposes on account of the great advantages it offers, some of which will become evident from the following brief account.

A point worth noting is that the design and manufacturing characteristics of these structures make them suitable for use on a European scale.

The "Mobilpass" overbridge

General features

The difficulties resulting from the constant increase in road traffic can only be overcome satisfactorily by planning a network of motorways which are systematically connected to both town and country main roads by adequate intersections and multilevel junctions. There is a definite preference nowadays for elevated roads because they are easier and sheaper to build than subways and underpasses.

A suitable plan for connecting town and country roads to highspeed motorways can be worked out by close co-operation between the traffic expert, the civil engineer and the architect designer, but the actual implementation of the overall programme is a more difficult matter. The expenditure involved unfortunately makes it essential to allocate priorities and stagger the execution of projects over a long period. The time lost through bottlenecks and continuing congestion at certain points nullifies the advantages of long stretches of well-designed road. Demolition work, road-construction and maintenance operations cause delays and diversions. Often fully completed sections of motorways remain closed because work at some points is lagging behind the rest. Towns are cut in two because railway yards are shortly to be dismantled, and in other cases the collapse of foundations or other natural hazards halt traffic completely for a while.

It is to overcome these difficulties that a standard temporary flyover known as the "Mobilpass" has been designed and patented. Being of standard construction, it can be delivered at short notice and erected without alterations on a wide range of bends, levels and gradients. It can be assembled and dismantled without resorting to costly site facilities, and can be broken down into light and easily transportable components, all of which can be re-used.

These flyovers are thus an ideal means of dealing with urgent, but temporary road problems, either as a provisional but immediate remedy pending large-scale works at a later date, or as a component part of a road system where the ultimate facilities have still to be planned, approved and financed, or lastly, as a means of studying traffic flows in planning a new road project.

The "Mobilpass" flyover can also be used as a normal temporary bridge, as a gangway for heavy service requirements or as a landing stage. Its characteristics also are such that it can remain in position as a permanent elevated road.

Details

The "Mobilpass" has a carriageway 3.50 m. wide and is thus one-way only; if two-way traffic is required two structures are erected side by side.

The structure supporting the roadway deck, and the piers, girders, ties and bracings consist of steel sections.

The girders are standard and are designed for a 12 m. span but can also be supplied for spans of 6 m., 8 m. and 10 m. for use on sharp bends.

The piers are made up of three vertically-extending telescopic sections which can be adjusted to virtually any position desired. This enables any abruptly rising level from 0 to 5 m. to be covered. The tops of the piers are capped to take the ends of two adjacent girders with intervening bearing blocks. The latter are designed so as to accomodate both horizontal and sloping girders, whose arrangement can be either rectilinear or angled.

The gradient can be selected at will, the normal figure being an initial 1 in 28 sections followed by a final 1 in 14.

The approaches at each end consist of asphalted gravel ramps retained by reinforced concrete walls.

A guard-rail runs along the entire length of both sides of the "Mobilpass" including the two ramps.

The deck consists of parallel hardwood planks, suitably treated, laid crosswise and bolted to the girders.

The foundations are standard plinths to which the bottoms of the piers are secured by holding-down bolts.

The erection of the "Mobilpass", from the time the foundations are ready to the point at which the bridge can be opened to traffic, takes only a few days, depending on site and traffic conditions.

The interruption to traffic caused by the erection of the central span lasts only an hour, but on the average is considerably less.

The "Mobilpass" can be supplied with one or two pedestrian footbridges.

The outstanding features of the structure can be summarized as follows :

1. Maximum speed of erection without disturbance to traffic or provision of site equipment.
2. Flexibility of application to a wide variety of gradients, bends, spans and carriageway widths.
3. Lightness and ease of transport.
4. Reduction in pier clearances and foundations costs.
5. Attractive appearance blending with urban surroundings, even when intended as a permanent structure.

Road and railway footbridges

Introduction

Pedestrians nowadays find it increasingly difficult to cross the road. Devices such as zebra crossings and traffic lights are inadequate and cause traffic to slow down, leading to hold-ups.

Subways and underpasses have been built in many towns and cities, but by comparison with overbridges their construction is slow and costly, and entails excavation, timbering, drainage work and interruption of traffic.

The problem of road crossings is not confined to major cities. Access to beaches at holiday resorts is often extremely hazardous, apart from holidaymakers having to wait a long time before they can cross the road. Towns and villages may be bisected or outlying areas cut off by new road and motor way intrusions.

The most satisfactory way of dealing with these various problems is to provide low-cost footbridges in the form of sturdy and attractive steel structures which can be erected without disturbing road or railway traffic.

Steel footbridges offer the following advantages :

— suitability for a wide variety of crossing conditions over roads, railways, waterways and canals;

— attractive appearance blending harmoniously with the surrounding landscape or urban environment;

— economy of land use, since steel constructions take up less space than concrete structures;

— ease and cheapness of transport even to awkwardly located sites;

— speedy assembly without interruption to traffic and without the use of scaffolding, in any weather conditions;

— absorption of sub soil movements;

— economy in manufacture and use owing to the mass-production of identical and interchangeable components;

— facility of dismantling, removal and re-erection where necessary to meet new planning requirements;

— opportunity for cheap and symmetrical erection alongside road overbridges.

Dimensions of standard footbridges

The footway width of various types of standard bridge already in use at a number of places is 1.50 m. The maximum heights are 4.75 m. and 6.75 m. for road and railway purposes respectively. The spans are 10, 14, 17, 22 and 27 m.

The bridges consist of girders simply supported at each end of the span, with projecting landings 1.60 m. wide to which the approach ramps are secured.

In urban areas the standard versions can be easily adapted to any local conditions and purposes. The above spans cover all ranges of motorway, trunk road and local road widths. The maximum-width version for railway crossings enables at least seven parallel tracks to be spanned.

Stairways, piers, parapets and foundations

The access stairways consist of C-section balustrades and steps of embossed steel plate, there being two flights with a steel landing halfway up.

With the railway-type of footbridge an extra bottom flight is provided to add an additional two metres' height, the remaining two flights being the same as in the road version. The stairs can be attached to the bridge either on its axis or at right angles, and in either case the two flights can be arranged in line or parallel.

Thus it can be seen that these designs, with their variants, enable the standard footbridge versions to meet all possible requirements.

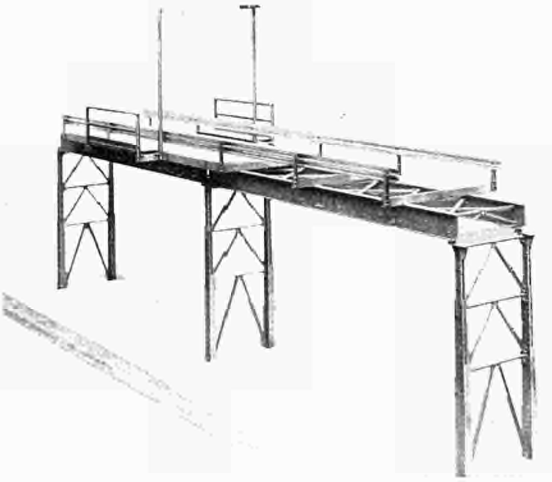
The parapets are of pre-welded tubular sections 1.10 m. high for urban road crossings and can be fitted with a protective screen 1.90 m. high for railway and motorway crossings.

The type of *foundation* required has been carefully designed to meet the majority of conditions found in practice. Each set of piers has its own plinth cast on a bed of lean concrete.

These structures can be transported without requiring the use of special vehicles; their components, which take up little psace, can be delivered loose to the site and bolted together on the spot.

Appearance

When the simplicity and functional quality of the well-proportioned structure, with its uniform girder height and graceful tubular column design are supplemented by an appropriate layout and a suitable colour for the paintwork, this type of footbridge stands out as an attractive landmark in its surroundings.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Siegfried KRUG

Assembly of Plastic-Bonded Decking of Footbridge

(Original language: German)

A rather unusual combined pedestrian and electric truck bridge was constructed last year in Rheinhausen (1).

This bridge has truss-type main girders which serve at the same time as the bridge rails. The decking is of precast reinforced concrete slabs, and cross-beams, bracings, etc., have been dispensed with altogether. The lower slabs were prestressed longitudinally with the bridge during assembly.

The bridge is of particular interest on account of the use of plastic adhesives as jointing mortar, and of the special assembly methods for prestressing of the concrete slab.

Figure 1 shows the design of the bridge. The main girders initially connected only the two terminal slabs. The connection was secured by high-strength prestressed bolts and plastic mortar with epoxyde resins; when the resin had hardened, the rest of the slabs were attached below the main girders, an air space being left between the lower chord of the girders and the slabs. Initially there were also interstices between the slabs; these were filled with epoxyde resin, thus jointing the slabs. Connecting reinforcing steel teeth about 5 cm. in length projected into the scallop pattern of the interstices (2).

After the plastic mortar had hardened (which took only about 24 hours), the whole was raised slightly on a centre support, the slabs being prestressed longitudinally with the bridge. The stresses involved are indicated in figure 1. The advantage of the method is that the pressures in the concrete slab are constant almost up to the ends, and not triangular, as otherwise tends to be the case in prestressing owing to movement of the abutments (shows as illustration at the bottom of figure 1.)

After assembly, the spaces between the slabs and the lower chord of the girders were filled up, and finally, when the adhesive had hardened, the high-strength prestressed bolts were tightened.

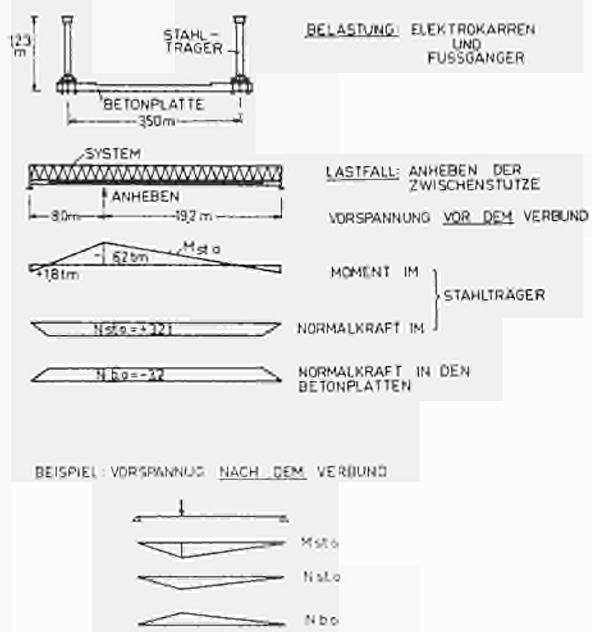


Fig. 1 — Sizes, diagrams showing bending moment and tensile stress as a result of slabs being prestressed (the whole raised on centre support).

Traffic on the road below the bridge continued without serious interference during the two-day building operation (3).

The main purpose of the exercise was to study certain practical points, such as

(1) assembly of precast steel and concrete slabs;

(2) employment of artifical resins for statical and constructional functions;

(3) possibility of erecting this type of composite construction economically;

(4) avoidance of disruption of traffic during building.

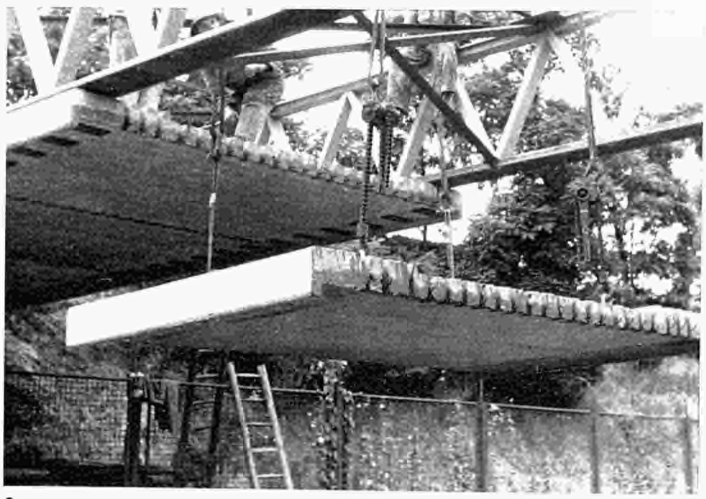
The results may be considered satisfactory, and other studies will follow. Full advantage is not yet being taken of steels potentialities (due to the assembly and erection methods normally used in steel construction) as regards avoiding obstruction to traffic, and it is important that these should be appreciated, especially by those commissioning the projects and by the planning authorities.

Description of photographs

1 — Combined pedestrian and electric truck bridge in composite construction with truss-type girders serving as bridge rails and precast concrete slabs as decking. Design measurements and lines of bending moments and normal forces under load. — Prestressing by slightly raising centre support.

2 — Positioning of precast reinforced concrete slab. Connecting reinforcing steel teeth about 5 cm. in length project into the scallop pattern of the interstices.

3 — Traffic on the road below the bridge can continue during the assembly operations which are of short duration.



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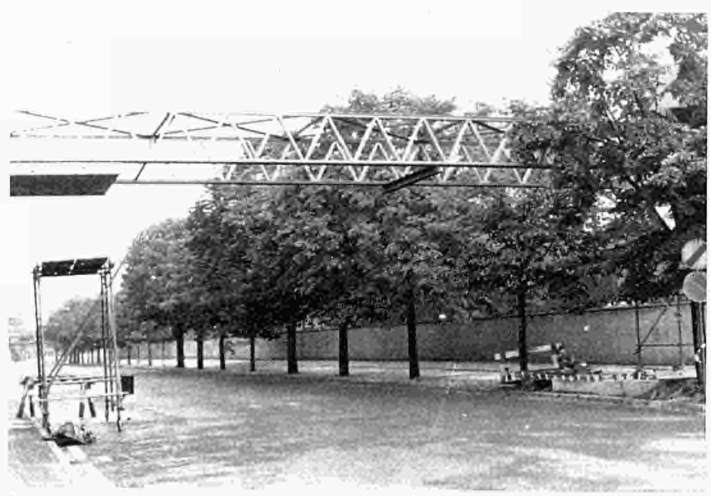

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Findings

Within Working Party II a number of viewpoints and suggestions were formulated which appear to be of sufficient interest to be set down here as findings. The High Authority should examine these and strive to implement them within its sphere of competence.

(a) Bridge- and highway authorities of the various towns and countries should be informed that the use of movable steel road structures would enable road traffic to be substantially improved.

(b) It appears advisable to stimulate the dissemination of information in this field by means of publications. There exists, it is true, a sufficient number of technical and trade journals, but there would appear to be further possibilities for also reaching architects, administrative departments, etc. In this connection the emphasis should be not so much upon the technical design and details as upon the economic usefulness and the effect achieved in the eyes of the community. Also, it would be advisable to organise excursions that would enable participants to see outstanding structures of this kind. Such excursions would be intended for representatives of the public authorities, planning departments and general public rather than for steelwork engineers and contractors.

(c) Each time road construction work in towns or on trunk roads is contemplated, not only the actual cost of construction should be taken into account, but also the cost to the road users which such work entails throughout its duration. Estimates that have, so far, been made in connection with such investigations show that this latter ccost can sometimes work out several times as high as the cost of the actual construction. The same applies to the cost arising from accidents, and this could be avoided by using sufficient guiding and safety devices. From the viewpoint of the national economy, substantial savings can be effected in this way.

(d) With regard to traffic signals and signs, the completest possible unification and standardisation should be undertaken, in order to reduce the manufacturing cost and thereby promote their use on a much larger scale.

(e) From the proceedings of Working Party II it becomes apparent that, in very general terms, it would be necessary to carry out systematic investigations into the maintenance costs of steel structures and structures of other materials, in order to arrive at a correct assessment of these costs, according to the various construction materials concerned.

It can here be stated that, in all probability, the steel structural components examined by Working Party II will in the future find new applications and that the scope for such applications will be further increased by the needs that arise. Therefore this field of application should benefit by particular attention and encouragement.

WORKING PARTY III:

Structural Steel Framework

Chairman :

James RUDERMAN

Rapporteurs :

Duilio SFINTESCO

Prof. Leonardo ZEEVAERT

Dr. Curt F. KOLLBRUNNER

Working Party III examined the possibilities in the field of metal frame construction, with special reterence to the technical and economic advantages of steel. It appeared that most problems relating to corrosion, fire and wind effects had been successfully overcome. As regards jointing, an important element in the cost of a metal construction, automatic welding seemed to be coming more and more to the fore.

There was still scope for further progress in a number of respects, including the use of new types of building components (tubes, bending-quality sheet), and the encouragement of new kinds of structure in metal building.

The Working Party considered it desirable that a study should be made of the possibilities for combining steel and concrete in the same building, more particularly in composite structures.

Duilio SFINTESCO

Structural Steel Framework — Today and Tomorrow

(Original text: French)

In this report that opens the sessions devoted to the subject of steel structures and frames for buildings, I propose to examine some common aspects of the problems that at present affect developments in structures of this nature. For this purpose I should like to discuss in turn and very briefly questions bearing firstly on the very fundamentals of the production of steel structures, namely, the materials, the construction and connections of structural members, together with technical regulations, secondly on the conditions that govern the stability of structures with special reference to their resistance to wind action, as well as to fire and rust. Lastly I shall deal with matters affecting the design of structures, that is, conventional forms and the search for new forms. Here I shall lay stress on the influence that technical developments, improved methods of calculation and, in large measure also, the growing use of electronic machines may have on structural forms. I shall conclude by sketching in outline the essential conditions in which structural steelwork will be able to succeed in the ever-widening field that lies open to it.

The programme I have outlined is quite an extensive one, but I think that an introductory address ought more or less to touch on all these matters in order to elicit the questions and comments that I hope will enliven our proceedings.

Structural steels and shapes of products

Steel production has always been a branch of industry that is well in the van of progress. Improvements still being effected in methods of production and in rolling technology give clear evidence of the dynamic nature of this industry.

The result is that constructional steelwork now has at its disposal the basic material with which it is possible to perform all sorts of structural feats, with a fair margin of safety to spare.

At present two main types of steel find their place in structural work: mild steel of Type A 37, which is represented in the majority of conventional structures, and high-tensile steel of Type A 52, which is finding increasing use, not only in bridge building but also in building the classes of structures with which this report is concerned.

The differences, however, between the two types of steel, both in the way in which they are used and in the economic factors that go with it, is so great that in some countries it is thought necessary to produce intermediate types. Thus we have, for example, in France, Type A 42 steel specified by our Bridge and Highway Department.

An investigation of the technical considerations that govern the production and use of a steel between the A 37 and A 52 types, strengthened apparently by economic factors, leads one to think that a steel which following the present nomenclature should be designated Steel 44, with a yield point of 19 t.s.i. (30 kg/mm^2) would have particular advantages for the progress of constructional steelwork. Such a steel, the production of which is being planned already in some E.C.S.C. countries, finds its application between Types A 37 and A 52, being, however, much closer in character to the first than to the second, because of the ease with which it can be produced and rolled. Hence its value.

A few more words about the varieties of steel. The need for having enough to choose from for any particular purpose is obvious, but it is essential to keep the number to the minimum. The few concessions, to which one must be resigned if this aim is to be achieved in so many fields of use, would be largely compensated for by the tonnages available, speedy deliveries and improved stockholdings. This is said with an eye to the programme of E.C.S.C. standards, now in preparation, for their drafting seems already to be running up against some difficulties.

Yes, it is obviously a good thing to have several types of steel to draw on, but to select them wisely for use in every case is still better. Structural steelwork engineers understand this and their European Convention has drawn up its recommendations in this matter.

I could hardly close this part of my address without mentioning a few kinds of steel products.

The series of "European" beams of Type IPE, first studied by the structural steel engineers within their European Convention, then brought into being in exemplary collaboration between producers and users, gives a clear proof of the progress that can be achieved by such co-operation. Even if for the moment the production of these beams is coming up against some troubles, such as higher prices, duplication of stocks, etc., it cannot be halted, for this is the law of progress.

Hollow sections, round, square or rectangular, which are available now in quite a wide range of sizes, find ever-widening opportunities for use in structures and frames for buildings. These tubular sections are specially applicable for struts, not subject to bending. They are eminently suitable for building the new types of structures.

Flat products, apart from the multiplicity of uses, which they owe in large measure to welding processes, will doubtless develop rapidly in the sector, which has a greater future, namely, pressed steel construction.

I have left to the end a question which is really the order of the day. We still have, since the last century, the rolled steel angles, with a thickness of one-tenth of their width, a ratio arbitrarily fixed by a predilection for round figures, but otherwise not justified in any way. It has been shown that the ideal ratio is in the region of one fifteenth, if the metal is to be most effectively used when the angle is in compression. For this reason, at our request and at our insistence, the G.T. 11 Committee of E.C.S.C. has included a series of thin-walled angles in the new Euronorm (E.C.S.C. standard). This will, however, mean nothing unless these easily rolled sections are adopted as standard by member countries and, still more important, unless they come on the market free of any surcharge which would outweigh all their advantages. Here then is an obvious matter for publicity directed at both producers and engineers, with the aim of enlisting both parties in this forward movement which means the better utilization of steel.

I would add that thin sections, having been systematically hardened in rolling, offer a high degree of mechanical strength. I hope the day will come when we are allowed to take account of this in our calculations, which will make them still more useful.

Fabrication and assembly of structural members

Two diametrically opposite forecasts are current about the construction of structural units. Some parties award the future to flat plate or sheet products, built up by welding and when necessary formed under the press. Others pin their faith more in the development of rolled sections. The first group bases its opinions mainly on the economics of automatic methods of welding and fabrication, the second on the relatively small amount of fabrication required by rolled steel sections. We think there is much to be said for both views. The two classes of units are complementary one to another for dealing with the vast field covered by our work.

Now, about methods of assembly and connections — welding processes offer and will continue to offer great potentialities in the design and execution of structures. This technique forms a permanent way of progress, inasmuch as it leaves a great degree of freedom in the design of constructional arrangements and enable the designer to depart from routine methods whenever the chance arises.

Connections made with friction grip high strength bolts represent a new technique which has already asserted itself with distinction, because of the rigid fixing it produces, comparable indeed to that of welded connections, and again by the ease and safety it brings to erection operations, requiring neither bulky equipment nor specialist tradesmen.

Another technique, not yet fully explored, is that of fixing with adhesives, which should lead to interesting results, either for connecting up secondary elements or for application to thin-walled units with contact surfaces wide enough for such treatment.

A combination of high-tensile bolts with welding, alternatively with adhesives, is also visualized.

Assembly with ordinary bolts, which has the advantage of being the cheapest method, still retains for itself wide enough scope in the erection of ordinary structures and frames. It should be added that the statutory French Regulations, based on experimental findings, especially in the matter of bearing on hole edge, favour the wide use of these bolts wherever suitable.

On the other hand, the once popular method of riveting is now out-moded and even on the way to extinction in some countries. Nevertheless in other countries it remains in favour in a big way.

Questions relating to the resistance of structures

In that part of my address which concerns the stability of structures, I shall deal first with wind, then with fire and corrosion.

In regard to wind, then. Of the numerous kinds of loading, the action of wind plays a leading part in dimensioning of structural members, its effect being quite decisive in some types of main or secondary units.

Wind action on structures has been the subject of important theoretical and experimental investigation during the last twenty years. An international conference of eminent authorities in meteorology and aerodynamics, which was held at Teddington, Great Britain, came to the conclusion, however, that much uncertainty still existed about the effects of gusts and momentary distribution of pressures on the wide surfaces of buildings.

Hence measurements of wind pressure have been take on the external wall faces of two very tall London buildings with reinforced concrete structures, simultaneous readings being taken of wind velocities. An investigation of this kind requires very complex and delicate apparatus.

Reverting more particularly to structural steelwork — it seems to us that, though the question may be considered clear enough for the majority of structural types, it does not appear that any experimental proof has ever confirmed the relative values of the stresses due to wind that are taken in the Regulations as a basis for the calculation of columns in multi-storey buildings. What happens is that these statutory rules start by taking wind velocities, then proceed by making a number of assumptions, some more or less proven, but others not at all, and finish with the high stresses which are supposed to be produced in the columns.

It would admittedly be a tedious business to seek an answer from the results of measurements taken on a single object so complex as a building under conditions of occupation. For my part, I think all the same that this matter is of such importance that an attempt, difficult no doubt, should be made, but a simpler one than that carried out in England, with tests that would be of direct value for structural steelwork. These tests would include the recording of stress variations in correlation with wind velocities for some columns in a suitably chosen building.

In spite of the scepticism with which such an effort would be viewed by some people and without reckoning to obtain through it any relaxation of current regulations, it nevertheless seems worthwhile to know once and for all whether, in a building of normal construction, the effective variations in the stressing represent, for example, one half or even one tenth of those that the Regulations assume.

In this connection I should here like to suggest that, if the rules that specify wind loading on structures have been arrived at in an objective and uniform manner for all methods of construction, such objectivity can only be illusory, inasmuch as any justifiable relaxation would automatically favour development in structural steelwork, just as at present excessive assumed applied loads of necessity favour methods of construction that are less affected by these loadings.

I would take this opportunity to say a few words about two arguments that are wrongly advanced against steel: its supposed inadequate resistance to **fire** and **corrosion**. Therefore I am including both matters under one heading, though they are otherwise so different one from the other. Referring to them recently in another context, I had no hesitation in calling them the "imaginary enemies" of steel.

Much ink has been spilt in dealing with the performance of steel structures when exposed to fire. One finds everything, from the most factual and most intelligent statements down to exaggerations and solecisms that have succeeded in creating a baseless prejudice in minds not sufficiently acquainted with the problem.

The approach, which seems to us the correct one, is to assess the hazards accurately and objectively, this being the only basis for a rational definition of suitable means of protection.

Without discussing at length a subject which Mr. Kollbrunner wil certainly handle with all his authority in this field, I cannot refrain from saying that, in spite of the second thoughts manifest in the recasting, recent or now in progress, of statutory regulations in some countries, including our own — and I here acknowledge our debt to the authorities in charge of this work — there still exist in some codes of practice prescriptions that call for useless and excessive protection and which seem to be motivated by fear rather than by a convincing analysis of actual hazards.

Here again we think that an experimental examination of actual conditions is required. We should obtain by this a statistical re-assessment of the fire loads that have in fact to be contended with in modern blocks of flats and offices, with the object of getting reasonable and realistic values for the statutory fire loads. We should also observe tests on finished buildings under fire attack and tests with real fires in buildings with their usual complement of furnishings and their steelwork unprotected or very lightly encased.

Such tests would supplement furnace tests whose scientific value is clear enough, but only allow one to study isolated aspects of the problem, these being singled out from the whole combination of circumstances that attend any real outbreak of fire.

Just a very brief word on corrosion. In certain circumstances iron rusts, as we know.All the same, our structures do very definitely have what you might call an "iron constitution", whether protected by very simple means, according to degree of exposure, or not protected at all where this is not necessary.

All in all, the problem of corrosion-proofing has been quite satisfactorily coped with. We shall be able to deal with it even better, and more economically, when descaled rolled steels will be supplied.

The design of structures

Classic forms and search for new forms

With the exception of the Eiffel Tower and maybe the „Atomium", a structure is never an end in itself. It has to serve some purpose and has therefore to be designed and erected to fit this purpose. It should be, as it were, a logical conclusion to this. Logic demands that one should also be guided in the design of a structure by the properties of the material used and by the technical means used to produce the building. It is natural then that the use of steel should have given rise to structural forms based primarily on its inherent properties of toughness, ductility, flexibility and above all on this material's characteristic high strength under all kinds of stressing: tension, compression, torsion, shear.

This explains why steelwork has never been a massive kind of construction, but one that tends always to assume slender forms, for lightness is inherent and quite natural to it and boldness is its very essence. Another fundamental point is that a steel structure is always built of prefabricated units, this in spite of the novelty of the term, "prefabrication". The idea is inseparable from steel construction and may be regarded as a characteristic feature. Moreover, the devising of structural forms is of necessity influenced and conditioned by the means available for assembling its component parts.

Steel structures, whose general design was originally suggested by timber construction, obviously not by masonry, arrived speedily at what we now know as "classic" forms. The outcome of this evolution was not stagnation, but stabilisation at a point where it made possible the erection of numbers of noteworthy constructions.

Each of the bars or plane elements that go into these structures is required to perform a distinct function of strength or stability, or at least this is the aim of the simplified theories which underlie calculations for them. Nevertheless this is not even quite correct in the case of systems so designed, and this is one of the reasons why we cannot help finding in such work an excessive margin of strength. This entails a waste of metal, or, at best, room for economy.

From such observations it was quite natural for structural steelwork to get gradually out of the rut and to tend often to move away from isolated elements, performing a single and specific function, towards the conception of an integrated structure, capable of taking conjointly and indistinctly all kinds of stresses.

Such a departure represents moreover a vital need for structural steelwork, in that it uses to advantage all that the properties of steel have to offer, retaining nevertheless in the finished product a measure of simplicity and economy that does not cancel out the benefits that are to be realised. We would make it clear that this concept can be applied to systems that use solid plates, to box sections, for example, with their strength in torsion, as well as to reticulated systems, such as three-dimensional or space frames, or to self-supporting or suspended shell structures, and so on.

This development is found, for example, very clearly in the case of wide or medium span roofs, the many varieties of which range from the suspended shell roof, a very suitable form for a material as strong in tension as steel, to the domed roof, traditional in the first place, but now become the rightful property of steel, by virtue of its construction in flat units or slender bars. Developments such as these have made possible throughout the world the construction of a number of halls of all classes, which mark a step forward in contemporary architecture, their fitness for purpose being equalled only by their aesthetic merits.

The structure of these roofs consists usually of a network of light members, either in pressed steel or often enough of tubular sections. In passing it should be noted that a tube, which is most efficient when stressed along its longitudinal axis, lends itself admirably to joining up members that run in diverse directions, and this makes it a simple matter to form the nodes in space frames. Turning now to suspended roofs, unless these happen to be built in steel plates or sheets as was the case with a very interesting circular industrial building of Austrian design, the element that recommends itself most strongly is the high-tensile steel wire rope.

Various types of roof, whether proprietary or otherwise, have made their appearance over the last ten years. These are in many instances influenced by the forms found in nature, animal, vegetable, mineral, for it is always nature that invents the most logical forms.

A popular shape for this new family of roofs is the parabolic hyperboloid, whose straight generating lines can be conveniently connected with straight perimeter beams. This form of roof, accomplished with the aid of a system of tensioned supporting ropes, all suitably prestressed, has been the method of construction of some notable buildings. I have too many personal recollections of the roof over the French Pavilion at the Brussels Exhibition, to miss taking it as an example, but it is only one out of a thousand perhaps.

Much has been said about the shapes of wide span roofs, which have always been a coveted field for the creative genius of the architect and for the art of the engineer, who is there to help him. On the other hand, another very important class of work seems to us to be rather backward in this respect, this is the steelwork for multi-storey buildings.

If one compares the structures of fifty years ago, of twenty-five years ago and those of today, it will be found that steel is used now at much higher working stresses, that new methods of making connections — welding, then high-tensile bolting — have come on the scene and that, sometimes for technical reasons and sometimes for economic reasons, the forces due to wind are taken either by rigid connections, by rigid tranverse frames or by windbracing. In short, no fundamental change has taken place.

It is true that there are some new features in the construction of the members of these structures. Beside the wide-flange beam that retains its place for members subject both to thrust and bending, we find the hollow section, built up by welding, and more recently round, square or rectangular tubular sections. A new technique, that of tubes filled with concrete, has also yielded remarkable results.

It must, however, be conceded that nothing has changed in the overall pattern of structures; always the same stanchions, the same beams and joists, whether continuous or in single spans, with or without cantilever. Can there be nothing left for us to look forward to, or will our zest for research and our imagination be unequal to achieving it? I do not think that this is the case at all.

It is hard perhaps to agree that one could radically modify the arrangement of such a structure which seems to be the natural form for the skeleton of a volume built up by placing parallelepipeds side by side or one above the other. For all that, it is not prohibited to imagine that the genius and creative spirit of engineer and architect, working together, might discover and open up new and unexpected possibilities.

Even now one can take stock of certain American structures which prove that this quest for new forms is not a meaningless word. I would mention a recent 14-storey building in Pittsburgh, the structure of which consists of a network of sloping members set on the four external walls, also the project nearing completion of two enormous towers in Manhattan, also with an external steel structure, consisting of immense welded hollow columns connected by great panels of steel sheet. In both of these cases the structure is literally a girder of box section, with a fixed end to the ground. In the first example, even the semblance of beams and stanchions has been discarded, as has in the second example the idea of connections between beams and columns, and the structure so formed serves also as the external envelope of the building, dispensing with the need of anything like curtain walling.

We should note in this connection some remarkable advantages of adopting types of structures like these: doing away with internal columns, wide-span floors built of latticed steel units, protection against fire easy to ensure.

Nevertheless, these structural forms, born, not of a mere desire for change, but of the wish to exploit the potentialities of steel to the utmost, may be only a cautious first step in the discovery of the new techniques that ought to breathe new life into building with a material the resources of which are yet far from being exhausted.

Effects of technical developments and advanced methods of calculation

It is now a matter of history that welding processes have exercised an enormous influence on the design of structures, opening up what were once unsuspectedp ossibilities. But every new method, every advance in this or that technique enables the structural engineer to go further ahead with his art. The most simple instances are often the most decisive. Thus, the castellated beam could not exist without automatic cutting and without welding. Some types of connection in structures could not be thought of before the advent of high-strength bolts. Structural forms which we find here and there today and shall doubtless see tomorrow will be hardly feasible without the use of adhesives.

One branch of structural steelwork that has not yet been properly explored is that of prestressing. Only Belgium and more recently the U.S.S.R. have turned a little towards this technique, which could without doubt be developed further.

Turning now to structural systems, I can still remember the days when, in some cases, people hesitated about adopting systems whose statics were too complex, in case the design costs should exceed the value of the extra metal that could be saved when compared with a simpler system. One was also inclined then to make do with simplified calculations, which made only a timid approach to the precision that the conditions demanded.

Who could imagine this state of things nowadays, when we have at hand perfected methods and calculations can be done in a twinkling and at small cost in electronic machines?

The use of this robot relieves the engineer of the drudgery of mathematics, which formerly took up his time and distracted him from the essential business of his profession.

Again, it is recognised that the theories underlying calculations only represent a more or less simplified picture of real conditions. Nevertheless, in the past was not one tempted to introduce hinged joints merely to simplify the statics and, in so doing, the calculations? So, to paraphrase Henri Poincaré, "There are no solved or unsolved problems, but only problems more or less solved." I would suggest that joints are only more or less rigid so that it is often correct to calculate them as such, for which purpose, I might add, our present methods of calculation are adequate.

These save us the need for hinged or pin joints, which are costly and generally quite useless.

Lastly, our methods of calculation should be adapted to the capabilities of electronic computers, in order to make greater use of these. There can be no question about this really. All the same it is just as natural to give preference to systems and methods of construction that favour the most extensive use of automation in workshop operations.

I cannot help saying here that welded construction in plates or sheet seems less suited to these developments than the everyday systems that use rolled sections in riveted or bolted construction, though these are otherwise treated as being out-of-date. I am acquainted with striking cases of structural firms whose success bears out the truth of this statement.

Conditions for the future development of structural steelwork

Ours is an age of very rapidly developing technology, with a corresponding transformation of the conditions in which man lives. Any technical process that does not fall in line is bound to find itself on the losing side.

These developments set up problems of an economic nature, that any industry must tackle, failing which it will go under.

It is necessary then for structural steelwork, even more so than its competitors, to meet the ever-growing and ever new demands of its users, while remaining competitive in regard to costs.

These demands may be summed up in two words: beauty and fitness.

Though the world of today may appear to be preoccupied with material questions, it is nonetheless sensitive to appearance in its environment. We use a material which, last century, was enough to arouse the irony of a Paul Séjourné — well known here in Luxembourg for his admirable bridge in masonry. We have, however, a material that appeals to the architect of today as being the most suitable ally for developing his art, for embodying his most daring ideas, for producing forms, often original in character and fundamentally new and modern, designed to satisfy the craving for beauty as much as the demand for comfort and convenience and the higher standard of living and achievement of people today.

In regard to comfort and convenience — remember that we build around and for man. We have to provide shelter and give him what he most needs: air, light and quiet.

The openness that can so easily be obtained with steel-framed buildings has already put this material into a privileged position. It remains for the architect and the structural engineer together to make the best use of this situation.

When it comes to acoustic insulation, steel is an excellent material for buildings in comparison with monolithic construction. It is advisable to treat this as a very important matter. Insufficient attention to this point, or perhaps economy at all costs in choosing a scheme, can prejudice irremediably the functional qualities of a structure and bring discredit to a way of building that is quite blameless.

The conditions that influence competitiveness are most difficult to summarise, because of the complex factors that have to be taken into account. I shall here merely give a glance at them.

To build cheaply without prejudicing the qualities of the work is an obvious and elementary aim. The outcome depends on many factors, almost on all that concerns the design and execution of buildings.

To utilise steel properly is, on the one hand, a question of perfecting methods of calculation, but also and still more a matter of perfecting structural design, in which respect the art of the engineer is manifest and which are associated with the methods of construction in two ways, not only being directly influenced by these, but pointing out in their turn the direction in which these techniques have to develop.

The proper way to use metal is to consume as little of it as possible for the end in view, by exploiting all its potentialities. Steel producers have no quarrel with this, for they are fully aware that, the less the amount of steel that is put into any particular structure, the greater the amount that will be incorporated in the totality of our work.

Great economy could and should be effected in the operations of fabrication in the shops and in site erection, by adopting advanced techniques and by rationalising the organization of shops and building sites. Here again it is the task of the design and drawing offices to create the best conditions for economical execution. The designer should not restrict himself to the design of the finished structure. It is his business to plan the methods of execution to suit the means available and even to sketch out the stages of execution.

Whether a structure is more competitive or less competitive in character can only be decided by reference to the completed building, for this is what the client wants. It avails nothing to study the structure from all angles if the main building work and the fittings, finishings and services that go with it wipe out all economic advantages. It is of no use, for example, to devise a structure that is economical in metal if it has to be clad or camouflaged by costly methods. One is forced therefore to think in terms of the end product, ready for occupation, and to assume responsibility for as much of the total project as possible or at least the main building work.

This does not mean supplanting the other building trades, but purely and simply to take responsibility for that which vitally affects the markets for our industry.

In speaking of competition, one naturally thinks in the first place of concrete. Let us get this matter straight.

Steel and concrete are close relatives, but it is wrong to regard them as estranged relatives. If they have oftentimes to contend about some object in sane and fair competition, this is a fine thing, for it leads to research, to improvement, to progress.

It is not a matter of blood feud, but of staying in business, each in the proper place for the greater good of those who use their products. There is room here for both, working separately or together.

Indeed, these two materials supplement one another in perfect accord in the case of composite steel-concrete construction. This technique, though it may be recognised, has not yet been exploited sufficiently either in the matter of research or in actual buildings.

As a loyal servant of steel construction, may I say, in this city of steel, that I feel a better understanding could usefully be promoted between steel and concrete engineers. A symposium, or indeed a congress, why not, on the use of the two materials in combination, could with frank and friendly co-operation yield some very interesting and profitable ideas for future structural work.

A fundamental question that arises as much from technical as from economic considerations and affects the design and execution of structural work, is the life-span for which one should build. The considerations of probability that underly modern ideas about safety and security only make this question the more important.

In view of the exceptionally changing character of the conditions under which modern man lives and works, it would be absurd to attempt to build for eternity as one once tried to do, when monuments that still exist today came into being, many centuries or even millennia ago. It is essential to have a sufficiently precise idea beforehand of intended life-span, in the light of which one would aim to build to reasonable standards.

Any realistic forecasts of loadings and of stability and durability of the structure would depend in the first place on this. The cost, the deciding factor in the bulk of our work, is but a direct consequence.

I must close with these brief observations, of necessity incomplete, but which cannot be ignored if one is interested in the future of structural steelwork.

This future bears a very encouraging appearance, since it is, to a large degree, entrusted to research, whose objectives must be decided by it.

Research, essentially a measure and indication of the vitality of a profession, gives the key to long-term development. This is a thing that European engineers have properly grasped, because they have come together for a Convention the main objective of which is just this, to increase the effectiveness and scope of the research which is their concern, by sharing in common all the means for investigation which they command and to direct this research into practical and realistic channels.

We witness nowadays constant improvements in performance in all fields, even where unexpected as in the performances involving physical activities of man — the repeated toppling of athletic records bears this out.

We see no reason then why steel, the material for great feats in building, should not retain the lead that it already holds and go forward with it to new heights in the development of man's achievements.

Leonardo ZEEVAERT

Structural Steel Building Frames in Earthquake Engineering

(Original text: English)

Introduction

The design of building frames in areas where environmental forces include earthquakes and poor subsoil conditions requires a special skill and experience from the civil engineer and architect to be able to produce a structural design that is compatible with such an environment. Seismic or Earthquake Engineering is a new branch of Civil Engineering dealing with the investigation and application of earthquake forces induced in structures founded on and in the ground; as buildings, dams, highways, railroad tracks, canals, tunnels, pile and pier foundations, surface and compensated foundations.

The proper behavior of any construction placed on the ground is highly dependant on its foundation, even for static forces, a well designed superstructure may perform a very poor work if its foundation is defective. Therefore, in a well balanced design there must be compatibility between the foundation, subsoil and foundation structure, and the superstructure supported by such a system. The present paper is prepared only to expose the design general features and behavior of structural steel building frames, therefore the discussion of foundations is out of the scope of this paper. However, the author will consider briefly subsoil conditions, since earthquake forces induced in the foundation of buildings because strong ground motions, play a direct roll in the design of structures to these environmental forces of nature.

The acceleration configuration of the ground surface during an earthquake depends primarily on the engineering characteristics of the subsoil [1] and constitutes a particular characteristic of a site or area where subsoil conditions are similar.

Steel, aluminium, reinforced concrete, plain concrete, masonry or timber and in certain occasions plastics, may be used to carry out a seismic design, in such a way as to produce a safe structure. The materials mentioned before have different characteristics of strength, stress and strain, and ductility. Therefore, the designer of a seismic structure should be aware when and in which occasions one or the combination of these materials should be used to design building frames with advantage and economy. The architectural design plays in this selection a very important and major part and therefore, from the very beginning of the layout and functional requirement of the project, the structural considerations must be considered from seismic engineering point of view in order to achieve the most favorable solution.

It is recognized that structural steel, because its ductility, is the best material that can be used in structures. Moreover, in case of tall and heavy buildings structural steel is found by experience most reliable and suitable for the design of a seismic structural frame. Nevertheless the design should conform with the dynamic forces induced by the earthquakes. The flexibility of the frame plays a very important roll. The design of a very flexible structure, in the aim to achieve maximum economy may not be suitable. A structure designed in such a way may stand perfectly the dynamic forces induced by an earthquake, however, architectural interiors, curtain walls and other expensive details may be damaged beyond repair. Furthermore, high flexibility in the upper floors of a tall and slender building may create the effect known as "whip action" with high accelerations, hard to take safely by occupants, furniture and other installations. Photograph 1 shows the collapse of the upper stories of a tall building in Mexico City because of whip action.

When a structural frame is underdesigned, stresses may reach values over the elastic limit, in such cases structural members deform permanently. The members themselves will not break because high ductility in steel. Therefore, in that respect, the building may be free from collapse, except when this happens in column members, where buckling may take place. Photograph 2 shows a steel frame failure during earthquake July 28, 1957 in Mexico City where one may notice the steel completely deformed, due to partial collapse of the structure because of failure of defective joints and compression members by buckling. Photograph 3 shows a two-storey structural frame in the verge of collapse because the columns were strongly deformed beyond the elastic limit, and within the plastic zone of the steel. When the structural members deform beyond the elastic limit, the repair may become a very costly proposition, and in many cases it is necessary to dismantle the building to be able to reinforce and straighten the damaged structural members.

If the foundation fails to work as expected, the superstructure will be submitted to unforeseen forces that may cause the collapse of the superstructure or large unrecoverable deformations. Photograph 4 shows this case where it may be seen that the failure of the structure was induced by a foundation subsidence.

From past experience in damage to buildings during earthquakes it appears desirable to design a building in such a way as to have the minimum possible damage, however, if damage has to be allowed this should be one that might be easily and unexpensively repaired. To achieve such a design it is necessary to limit the deformentional behavior of the structural frame during the earthquake, and to use stresses under the elastic limit of the material, in order to avoid the danger of permanent deformations. Therefore, flexibility of the structural frame should be carefully selected and relative displacements of the structure should be limited to values that may be easily taken in consideration in the design of the building architectural details. More important are the curtain walls, windows and rigid architectural elements like facings of granite, marble or any other costly materials (5).

In order to be able to predict that the structure will behave under the elastic limit it is necessary to learn on the earthquake acceleration that may enter the building during the most severe earthquakes that may take place in the area considered and for the probable life of the building. The use of old building codes is not always a good, economical and safe practice. Therefore, it is necessary to measure the real seismic forces by means of accelerographs. The information obtained from these instruments in the way of accelerograms may be processed by means of computers into spectra from where it is possible to predict with more accuracy the response of buildings.

Steel structures designed using above mentioned procedure will have a considerable reserved strength against collapse because of the high ductility properties of steel in contrast with reinforced concrete that having less ductility, in severe cases structures may collapse completely (6). However, structural members, especially compression members, should be carefully designed against buckling in the neighbourhood of the plastic range, joints between columns and girders should be designed in such a way as to be able to resist safely forces that may reach the elastic limit of the steel.

The literature at the present shows the destruction of buildings not properly designed in many cities of the world where there may be encountered few or no provisions in the codes for seismic design. The most spectacular damage has been reported in the earthquakes of Chile, Alaska, Yugoslavia, North-West Africa, Turkey, Mexico and in the past, California, U.S.A. and Japan, in the last two countries seismic engineering is developing rapidly.

Seismic design

In order to appreciate the advancement of seismic engineering in the design of structural frames for buildings, it is important to describe very briefly different procedures of design, namely the use of codes and the very recent use of spectra measured from earthquakes.

The use of codes or regulations in the design of buildings is extremely important and necessary to uniformize officially design criteria, and to avoid abuse in the design and use of materials. The codes are based mostly on experience and in the case of earthquake forces contain empirical rules that represent in general the expected average or even maximum structural behavior. No code may be considered yet complete and perfectly adequate for all conditions in seismic design. The principal reason is that in very few places of the world strong ground motions have been recorded and studied. Moreover, in those places where this has been achieved, either there is not yet sufficient information and the present state of knowledge is still in form of investigation and therefore, no definite conclusions may be set forth in such a way that they may be placed as regulations in City Codes. There is still very much to learn concerning the actual measurement of strong ground motions and the response of buildings to these forces. The result is that City Codes specify empirical acceleration coefficients to be used for structural design. When using these specified coefficients the structure may be easily designed by static considerations. Nothing is said about resonance or dominant periods of the ground and for definite and different subsoil conditions. Structural frames for buildings have definite dynamic characteristics that make them more sensible to one subsoil condition than to another. A multistorey structure has so many modes of vibration as floor levels. Although it is recognized that the three first modes of vibration may be the only important in the structural behavior, it appears that the higher modes may be also important in the building concerning secondary elements. The author has had the opportunity to observe loose partition walls at the upper floors of tall building in Mexico City, to have displaced over 100 cm. during an earthquake. The walls were not drameds in spite of this large displacement. Undoubtedly this could happen only by the vibration resonance of higher modes induced in the building during the earthquake.

Therefore buildings designed with modes of vibration close to the dominant periods of the ground are bound to be excited stronger than those buildings with modes of vibration away from that of the ground.

There is no doubt that when more information is obtained concerning ground motions in different regions and on the response of buildings to these motions, and also on the behavior of architectural details, the city codes of the future will contain precise regulations on this line. In Japan and California, U.S.A., studies have been made toward this goal, as well as in Mexico City, where new codes have been studied and prepared by a group of engineers at the Institute of Engineering, University of Mexico, who experienced earthquake July 28, 1957 in Mexico City. Experience and seismic measurements of Latino Americana Tower were used to certain extent to establish the proposed regulations reported at the Second World Conference on Earthquake Engineering held in Japan in 1961.

In order to illustrate the preceding discussion two accelerograms are presented in figure 1; the NS component of the strongest ever recorded earthquake at El Centro, California, U.S.A.[2] May 18, 1940, and Fig.2 the NS component for earthquake registered in Mexico City, May 10, 1962 [3]. Notice that the accelerograms of these earthquakes are completely different, showing El Centro ground accelerations about six times stronger than that of Mexico City; however, both of them show definite random impulses during the earthquake. El Centro shows very short impulses with periods on the order of 0.5 sec or less, corresponding the response on firm ground. On the other hand, Mexico City accelerogram shows rather large acceleration periods up to 2.5 sec, and is representative of the response of a lacustrine deposit 47 m. thick of volcanic clay [3,4]. Obviously these two different strong ground motions will produce different responses in a building with certain definite dynamic characteristics.

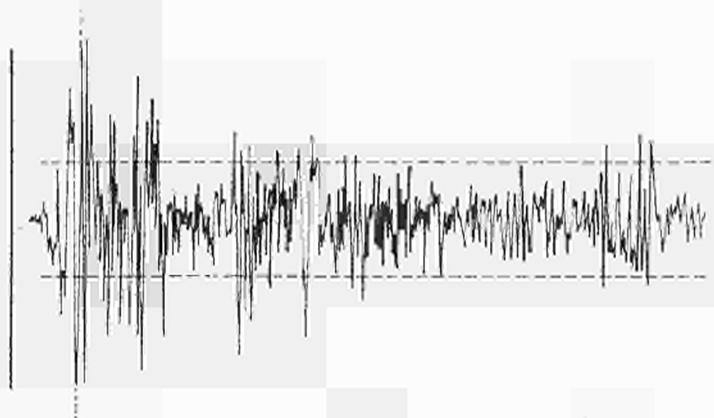

Fig. 1

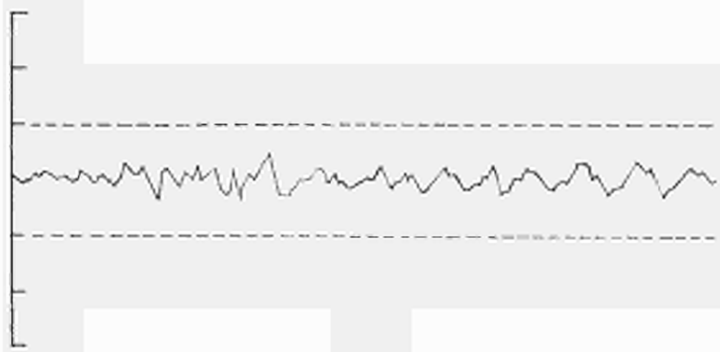

Fig. 2

The displacement response of a structure as a function of its period of vibration [5], is computed by the following equation

$$S_d = \frac{1}{\omega_d} \int_0^t a(\tau)\, e^{-\lambda\, \omega_1 (t-\tau)} \sin \omega_d (t-\tau)\, d\tau /_{max}$$

in which

$a(\tau)$ = accelerogram of the earthquake
λ = fraction of critical damping of structure
ω_d = damped circular frequency of vibration of structure with one degree of freedom
ω_1 = free circular frequency of vibration of structure
τ = time for certain acceleration of the ground
t = time at which the integral is a maximum

The integral term is usually called pseudo-velocity spectrum hence:

$$S_d = \frac{1}{\omega_d} \cdot R_g$$

The horizontal force produced in the structure of mass "*m*" will be

$$V_{max} = m \cdot \omega_d \cdot R_s$$

the value $\omega_d R_s$ is called in seismic engineering pseudo-acceleration to distinguish it from the absolute maximum acceleration:

$$S_a = \frac{\delta^2 S_d}{\delta t^2} + a\,(t)$$

Therefore the maximum force in terms of the pseudo-acceleration may be expressed as follows:

$$V_{max} = m \cdot R_a$$

The integration of equation 1 is impossible by hand and it has to be achieved by means of an electronic computer. The results for accelerograms in terms of pseudo-acceleration spectra, shown in figures 1 and 2 are reported in figure 3. From these figures it may be noticed the striking difference between them.

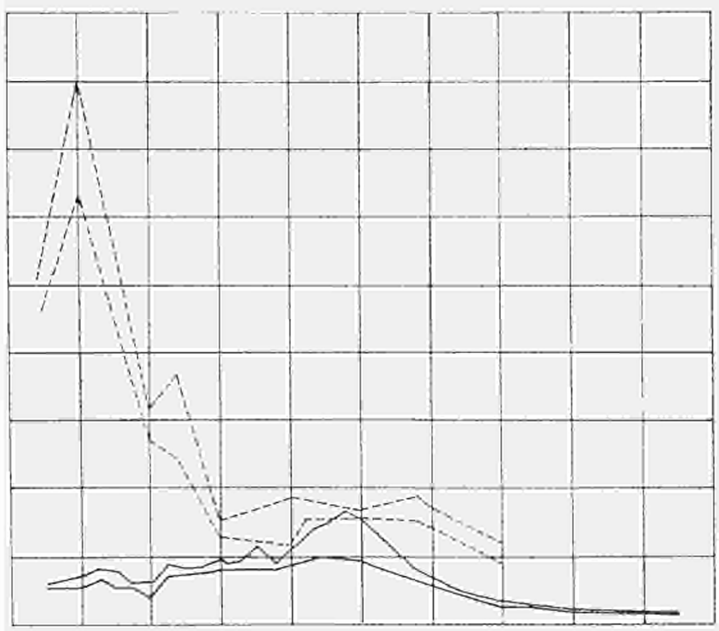

Fig. 3

El Centro, California, has a peak value corresponding to the dominant period of the ground on the order of 0.5 sec and that of Mexico City lacustrine area on the order of 2.5 sec. Therefore, a building with fundamental period of 2,5 sec will be stressed at El Centro only 50 % more than in Mexico City, in spite that pseudo-acceleration of El Centro shows much greater than that of Mexico City.

From above mentioned example it may be recognized that the rational procedure to perform a seismic structural design may be achieved when the displacement or pseudo-acceleration spectrum of the ground is known in conjuction with the dynamic characteristics of the building structure. It may be demonstrated that each mode of vibration of the building may be treated as an equivalent one degree of freedom system.

The response of each mode may be obtained from the acceleration or displacement spectrum and values thus obtained are combined to evaluate the most probable and maximum response shear forces taking place in the structure. After this is known throughout the building structure, shears, axial forces, bending moments, shear displacements between floors and total displacement may be estimated from stability considerations of the structural design. The flexibility may be investigated and also the most suitable resistant connections and structural member sections.

Steel in seismic design

In the dynamic design of steel structures it is important to recall certain important properties of steel, useful to the designer. Figure 4 shows a stress-strain curve for A-7 Steel according with ASTM specifications showing an ultimate yield stress at 2,720 kg/cm², however, for static design a tensile yield-stress of 2,340 kg./cm.² is allowed and about 1,480 kg./cm², for the shear yield stress. Dynamic tests " have shown that the dynamic yield stress is a function of time at which the yield stress is reached. For 10 seconds the stress may be considered as statically applied and no increase in the dynamic yield stress is observed. However, for one second it is slightly increased and for 0.1 second the yield stress is increased in about 10%. Therefore for steel structures with periods of vibration greater than 0.3 second the static yield stress may be used safely. After the yield stress is reached the strain increases at a constant stress slightly smaller than the yield stress and increases continuously up to 10 to 20 times the yield strain before strain hardening starts to take place. Therefore, the elastic work done in the elastic range is many times smaller than that necessary to reach the point of strain hardening. Therefore, a steel structure in its structural members is able to consume a considerable amount of energy before collapse, which is completely dissipated except for the part of the elastic energy stored in the structure. The ratio between maximum plastic strain taking place in the plastic range and the elastic yield strain is called the ductility factor. Therefore, the ductility is a measure of how much a structure may stand before large deformations may take place. The degree of deformation depends on the functional requirements of the structure when work in the plastic range is allowed.

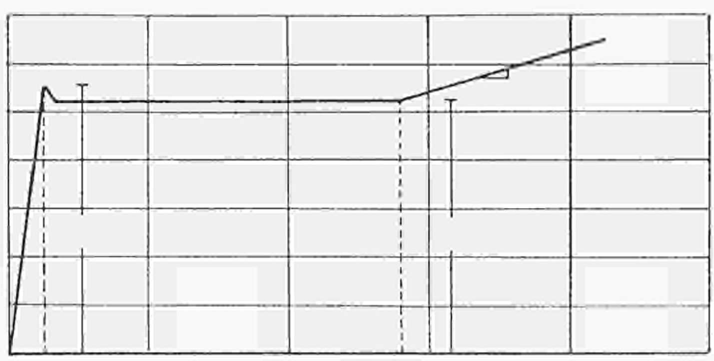

Fig. 4

In case of earthquakes when plastic stress is allowed and the structural members undergo reversal of stresses and strains (Fig. 5) a small plastic deformation will dissipate a considerable amount of energy with very minor or no damage to the structural members, provided these are designed against buckling. When displacements are close to the plastic limit but in the plastic range they appear not to increase considerably after the first cycle on account of the damping effect taking place.

In any event it appears to the author that concerning seismic structural design stresses and strains should be limited in structural members to those not exceeding the yield values. The structure designed in this way may stand in one occasion more severe working conditions without collapse. This may be translated in a larger capacity of the structure to take earthquake forces in the area under consideration.

Nevertheless, there are in a structural design many other elements that may be not in such favorable conditions as the structural members themselves. These are primarily the joints that have to transmit larger shear forces and bending moments. Riveted connections that are subjected to large plastic stresses may be

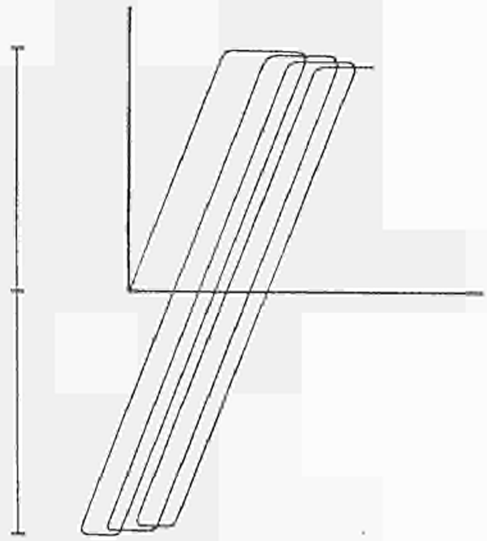

Fig. 5

strongly damaged, and may lose their rigidity to take properly next strong ground motion. It has been the experience of the author that steel structures poorly designed that have survived the first earthquake with relatively minor damage, but with large distortions in their girders, columns and connections, had to be rebuilt integrally after the third earthquake, and some have called for total reposition. Welded joints appear to behave better under high stresses as riveted connections except that welded joints may have high unknown residual stresses and in many occasions they show rupture at either the weld or in their welded elements. The crystallization of the steel in these cases should be watched carefully.

In general, it may be stated that the best practice is to limit deformations to avoid damage in secondary elements and architectural facings. Moreover, to obtain a structure with high reserved strength it is necessary to design joints and structural members against buckling and stress concentrations. Free plates in joints and flanges of conventional sections may easily buckle if not protected for this action, at points where stress may reach or not the yield values.

The earthquake forces may hit the structure with equal intensity in any direction; therefore it has been found that the most convenient column section is the box section either square or slightly rectangular, formed by four continuous plates welded together at their edges. Using this practice more economy may be found, and it is easier to design with the required moments of inertia, in contrast with the conventional wide flange column that is exposed easy to buckling and where the moments of inertia are quite different in both directions respectively (Fig. 6). Columns should always be kept in mind by the seismic designer as these elements are subjected to severe shear and flexure-compression work and stress concentration at the joints.

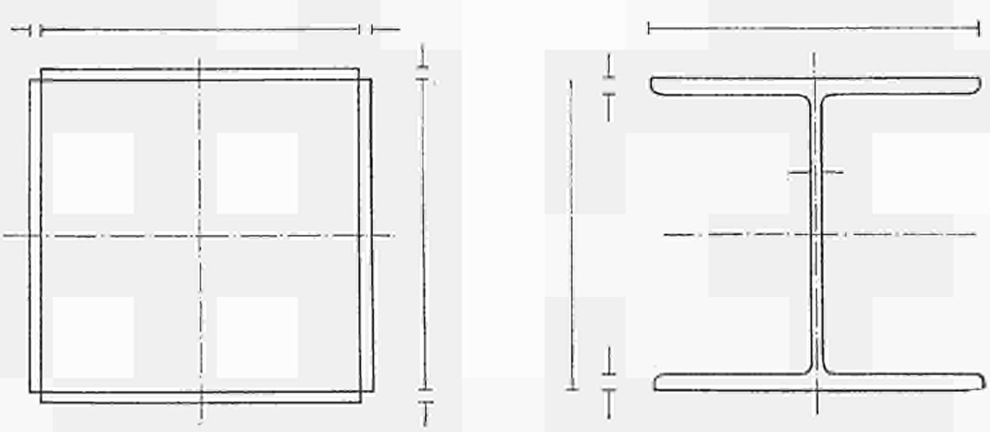

Fig. 6

The beams and girders that in conjunction with the columns give the necessary rigidity to the structure are elements that under high shear and flexure moments have to perform their work satisfactorily. They have to transmit high dynamic shear forces from one joint to the other in addition to static shears, their webs should be checked for buckling. The same may be said from the flanges, since in many occasions wide flanged beams buckle close to the joint. Therefore, flanges at the joints should be designed against buckling. A beam designed for dynamic forces will have considerable greater bending moments close to the support (Fig. 7). Therefore in order to gain economy it would be desirable to design these structural members of varying strength, that is with heavier section at the support. This may be achieved in two ways: (a) Adding cover plates to the conventional sections or the use of specially designed sections with flanges lighter in the middle and gradually heavier at the support (Fig. 8); (b) by providing more strength to the joint by means of haunches. The use of brackets is sometimes satisfactory although it may be in some occasions anti-economical, and more vertical space to locate them is needed.

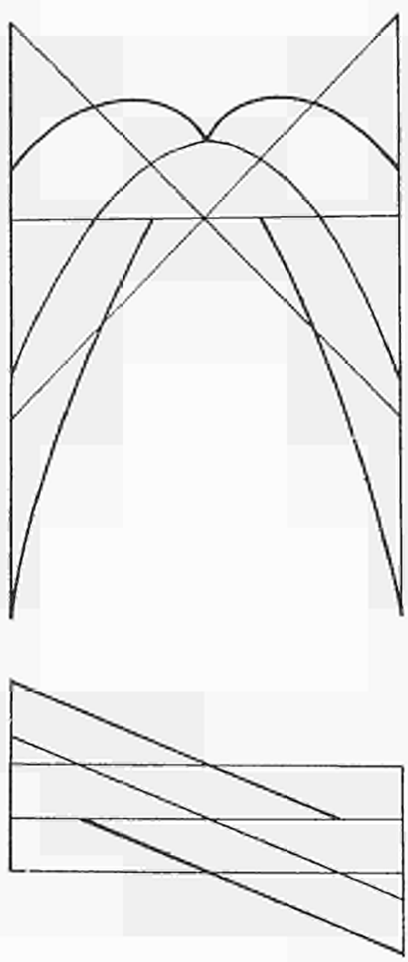

Fig. 7 — High shear and flexion moments of a beam

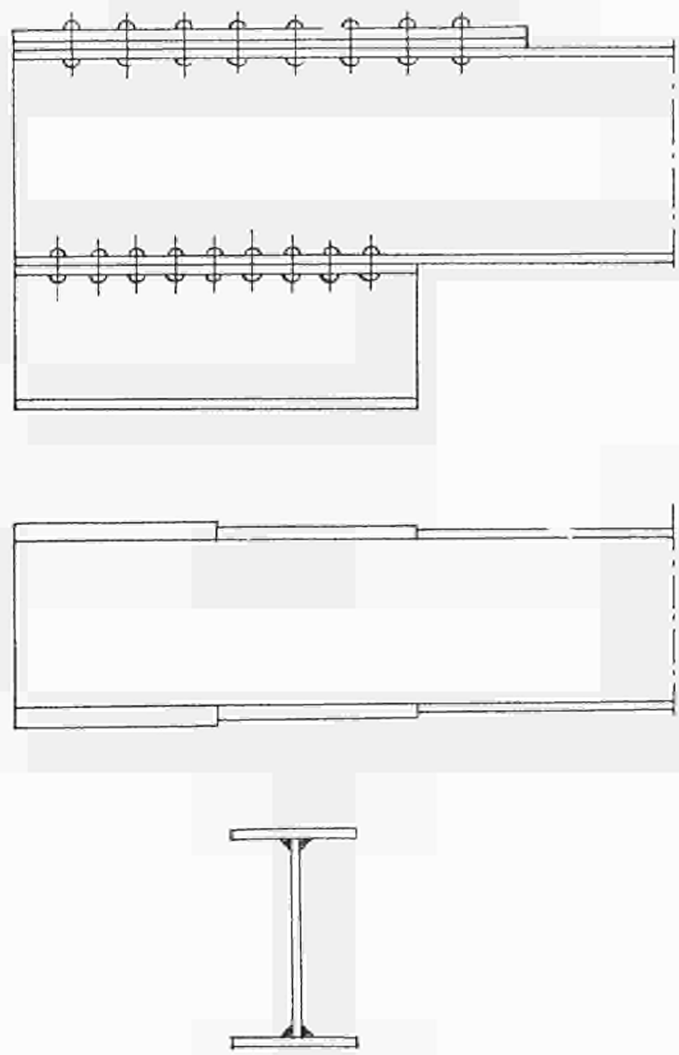

Fig. 8

The joints are by far the most important structural members of a seismic structural frame, as these members have the function of transmitting the mechanical elements between columns and girders, and therefore, hold the structure together during the action of these forces. The joints should be strong and capable to transmit such forces. Consideration should be given to stress concentrations taking place at the joints.

The design of joints should be made as simple as possible and should translate with clearness the way forces are transmitted from one structural member to the other. All possible indication of buckling should be studied and eliminated. Figure 9 shows different types of joints used by the author successfully in buildings described in further paragraphs.

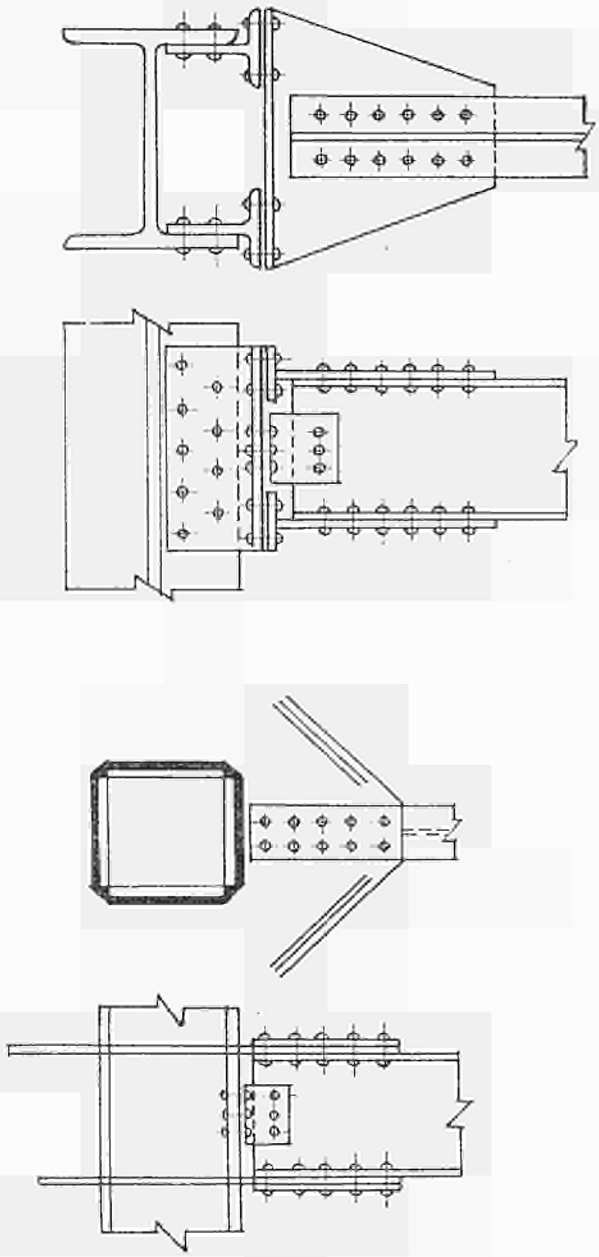

Fig. 9

When after designing a structural frame from the economical point of view, it is found that flexibility should be reduced for reasons explained before, it is then desirable to force the structure to work monolithically with the floor slabs. For economy floor slabs in general are designed of reinforced concrete. The connection of concrete slabs and girders is achieved by means of shear connectors designed to take the tangential shear forces that develop during bending.

Most effective are, to the experience of the author, channel sections, for the first time used in the design of Latino Americana Tower, to reduce flexibility without increasing weight of structural steel (Fig. 10). In the present other types of shear connectors have been introduced commercially as the "Nelson Studs" that are fixed in place by self-electric welding. The flexibility of a structural frame may be controlled to certain extent using this practice, and at the same time obtaining the maximum economy in structural steel.

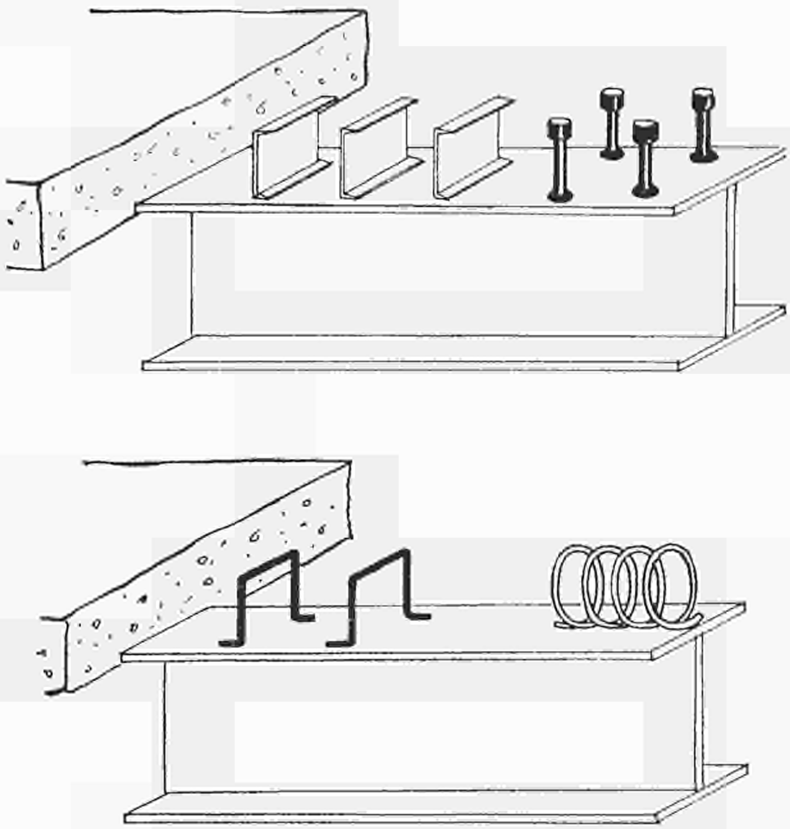

Fig. 10

There are always in a structural frame design stiffer bents than others. From the seismic engineering point of view, it is highly desirable that the floors behave as a unit. In major cases the concrete floor slab, or its way of construction, as in case of prefabricated elements is not strong enough to transmit in its plane the differential shear forces. For these purposes the structural frame has to be provided by means of diagonal members or diagonal reinforcement in or outside the concrete slabs to avoid diagonal tension cracking. It is desirable to have these diagonal tension members placed under an initial stress in order to avoid any retardation effect in their action. Post-tensioning in the concrete slabs may be used in certain cases.

The most important architectural details that should be seismically designed are the curtain walls and interior partition walls. Flexibility to these elements should be given to be compatible with the deformation of the structural frame as a whole and the relative displacements of the floors in particular. From the experience of the author, the most effective design of curtain walls is a steel frame with sufficient flexibility to follow the configuration of the building during dynamic action (fig. 11). The curtain wall elements as glass panels, stone or any other material, are attached to this curtain wall frame leaving in them enough clearance to avoid wedging action. Interior walls will be subjected to dynamic earthquake forces with high accelerations in upper floors; therefore, they shall be strong and firmly anchored to the floor they are supported [7].

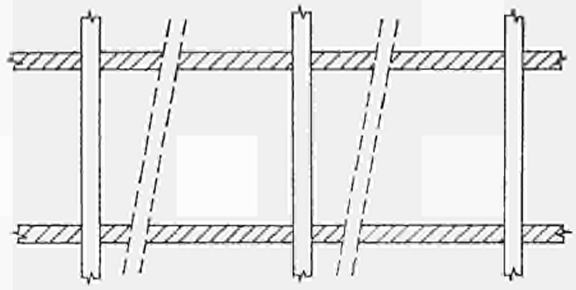

Fig. 11

However, since in most cases the partition walls are fabricated of rigid and brittle materials they should be designed as floating, that is to say: they should not wedge against the structural frame (Fig. 12). At the columns enough clearance should be left to take freely the floor displacement; and at the upper edge a sliding joint should be provided to avoid dragging forces generated from the relative displacement of the floors. One of the best practices is to fabricate interior partition walls with a steel light and flexible frame firmly anchored to the floor slab. In this frame the wall facing, insulation, etc., may be placed made of light-weight and compressible materials.

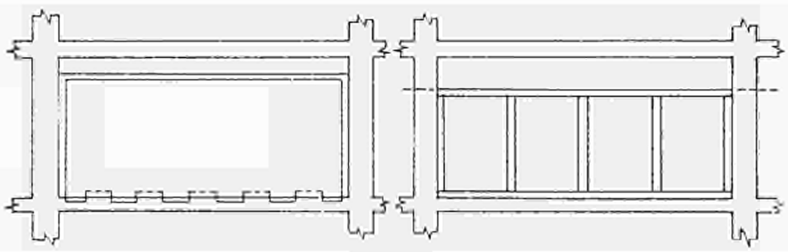

Fig. 12

Mechanical installations in the building like air conditioning ducts, electrical and sanitary conducts must be designed with sufficient flexibility to be compatible with that of the structural frame, and shear displacement between floor levels.

From above brief discussion on flexibility and compatibility between elements constructed in a building, the author has reached the conclusion that a more satisfactory seismic structural design may be achieved when the design is followed by limiting the total and differential shear displacements between floors, designing the structural frame compatible with the functional requirements of the architectural project, in comparison with a design on limiting stresses where deformations and displacements are considered of secondary importance. However, architectural seismic design should also conform with construction methods and materials that may satisfy the deformations and displacements limited in the design of the structural frame.

Latino Americana Tower

The Latino Americana Tower (L.A.T.) is an office building with two basements and 44 floor levels including the roof level at the observation platform at 140 m. from the street level and contains a radio tower extending another 42 m. from the roof level (7). The foundation of the building is designed as shown in cross section (Fig. 13) and works as a compensated foundation where the piles bearing at a depth of 33.50 m. on a compact sand stratum take only 50% of the total load of the building [8]. The balance of the load is taken by buoyancy. Figure 13 shows the subsoil conditions at the site the building is constructed. The subsoil consists of a series of very compressible lacustrine volcanic clays interbedded with alluvial deposits. The piles resting on the first hard sand stratum is however underlain by high compressible volcanic clay. The foundation is designed rigid with diagonal members to take care of earthquake torsion effects (8).

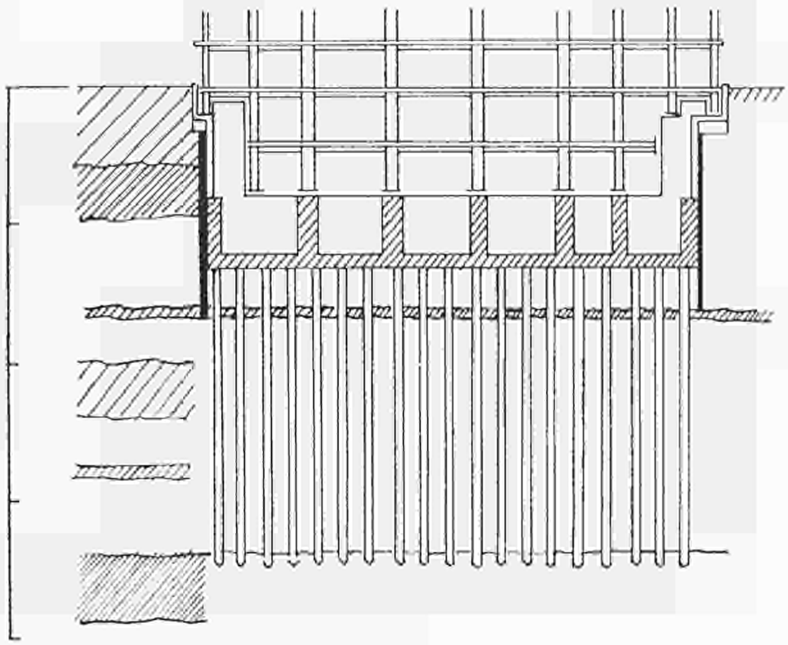

Fig. 13

The structural frame is designed of rolled steel sections. A general view of the steel structure during erection is shown in photograph 9 where the connections at the joints may be seen designed with split I-beams. The building structural frame was designed in 1948 using empirical experience from past earthquakes. The city code regulations for seismic design were not followed. These were considered out of date and inadequate. The design was followed for the first time with the use of a dynamic method that was compatible with the information obtained in those days in Mexico City [9]. The dynamic characteristics of the structural frame were determined to be the following values, first mode 3.66 sec, second mode 1.54 sec, third mode 0.98 sec, fourth mode 0.71 sec. The dominant period of the ground was estimated in 2 sec.

After the structure design was made, in its first step it was recognized that the structure was too flexible and the second mode of vibration was in the bracket of 2 sec., therefore danger of resonance and magnification of stresses was feared. Therefore, the structure flexibility had to be reduced. This was a costly proposition if steel sections were to be increased in size. Therefore in order to increase stiffness to the structure the reinforced concrete slabs were rigidly connected to the structural frame by means of steel channel shaped shear connectors. This practice reduced flexibility to about one half of the previously obtained. The prefabrication of the structure was ordered to Bethlehem Steel Corporation. In the meantime more studies from the dynamical point of view were continued, and the radio tower was added. The result of this investigation showed that the structure thus designed could develop in the upper floors "whip action" as flexibility still was too large in this upper section. Therefore, it was necessary to add stiffness to the already prefabricated and delivered structural frame members from the 28th floor to the top of the building. The stiffness was achieved by increasing the steel sections. The wide-flange steel columns were added with plates transforming them into box sections. The girders were added with strong brackets welded at the original designed structure (10). The connections were all welded at the floors where these reinforcements were made. Finally, the structure was obtained to be compatible with the expected behavior. The final dynamic properties are the ones reported before.

The curtain wall was made of aluminium and glass. The lightweight partition walls and mechanical installations were made compatible with relative displacements between floors. The floor slabs were added with diagonal shear reinforcement.

The building has passed already four major earthquakes, being the strongest the one that took place July 28, 1957 two years after the building was completed. The building behaved very well during these strong ground motions and very close to predicted behavior. Instruments have been installed in the building to measure shear displacements and in 1960 an accelerograph of the S.M.A.C.-B type, of three components, was obtained by the owners and installed in the basement of the building [3]. At the same time another accelerograph property of the University of Mexico was installed in a large Park about 600 m. away from the building. The displacement spectrum, N.S. component obtained at the Park for May 11, 1962, is shown in figure 3. The displacement spectrum obtained at the Latino Americana Tower is shown in figure 14. It may be noticed from these two spectra that the dominant period of vibrations is on the order of 2.5 sec. and is just intermediate between the first and second mode of the building. Furthermore, the intensity of the ground motion measured in the foundation of the building was about one half of that of the ground surface registered at Park. A more detailed report on the ground behavior and interpretation on observations made during earthquakes of May 1962 in Mexico City, and the instrumentation installed in L.A.T., may be found published elsewhere [1,3,10].

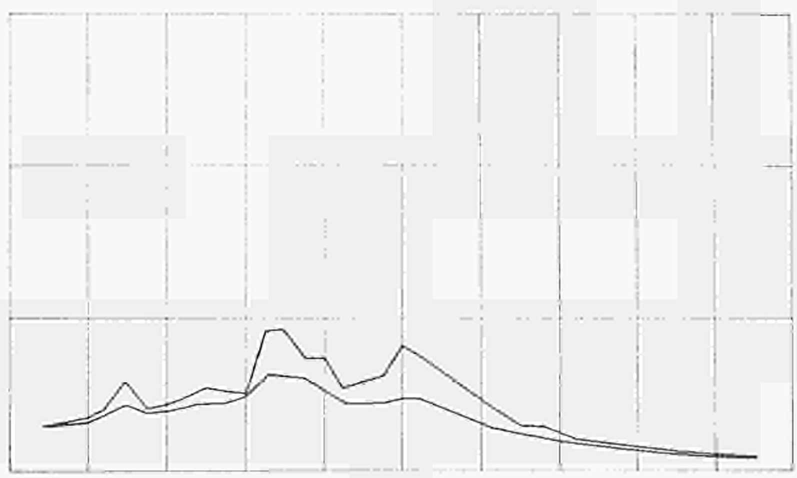

Fig. 14

The final design of the building was achieved with shear forces determined dynamically as shown in curve labeled A (Fig. 15); that represents the most probable shears that may take place including the participation of modes of vibration up to the fourth. The curve labeled B is the static shear as would be required by Mexico City Code before July 1957. The same figure shows dots representing the maximum forces measured in July 1957, during the strongest earthquake the building has experienced. Photograph 11 shows the finished building.

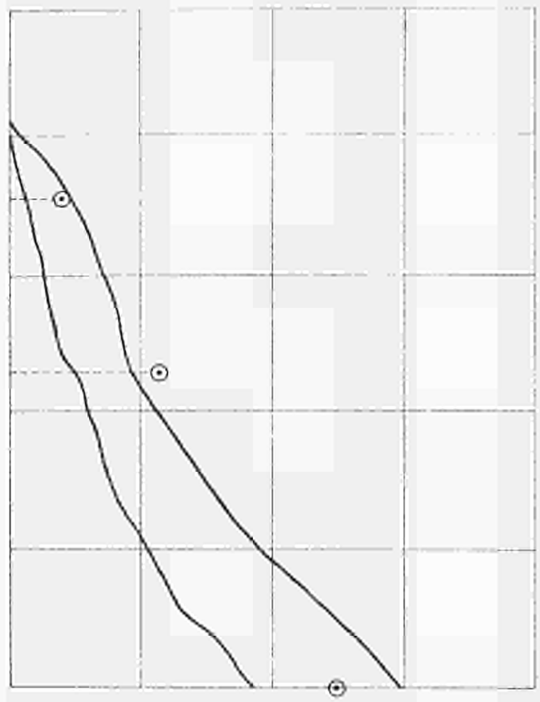

Fig. 15

Anahuac Building

The 23-storey Anahuac Building structural frame (12) is an interesting example of a prefabricated steel structure constructed with plates and angles riveted together (13) forming box type sections. The building was finished toward the end of 1956 just six months before the last great earthquake July, 1957 in Mexico City. The steel structure was totally prefabricated in Mexico City.

The layout of the Anahuac Building is shown by a typical floor (14) and cross section in figure 16, where it may be seen that the Tower mainly consists of two rows of columns separated 8.70 m. The height of the building from the street level is 81.2 m. Therefore, this building is the highest most slender structure in Mexico City with an approximate structural ratio of 1 to 9. Moreover, the building has only one axis of symmetry, therefore, in the lengthwise direction torsion is induced during earthquakes. Because of its slenderness the building turns out to be very flexible and therefore flexibility had to be restricted by using channel shear connectors to fix the reinforced concrete floor slabs to the structure. It was also necessary to design special joints called in Mexico City "butterfly type joints" as shown in photograph 14. Notice also that beams forming the rigid frame were designed of two prefabricated riveted steel channels, with stiffeners between them, obtaining in this way an effective connection and thus a rigid joint with the box type column sections. The plates forming the butterfly joint were provided with stiffeners at their free edges to avoid buckling during large compression induced in them during earthquakes.

A dynamic analysis was also used to design this building with the same methods as Tower Latino Americana, previously explained. This building was designed shortly after L.A.T. The dynamic shear configuration for

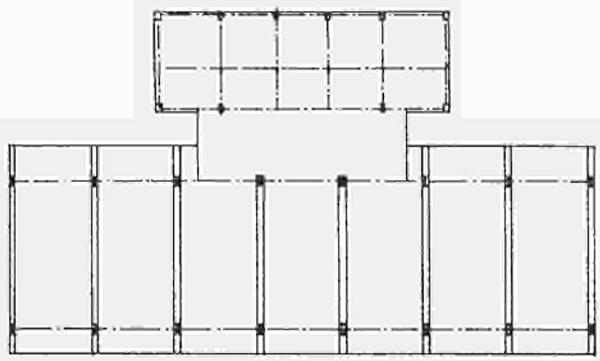

Fig. 16

design including the participation of the first, second and third modes is shown in Fig. 17. For comparison the shears required by the City Code before 1957 are also plotted; notice the striking difference. The fundamental period of vibration of the building is on the order of 2.2 sec. and second and third modes 0.8 and 0.5 sec, with coefficients of participation of 0.73, 0.15 and 0.08 respectively [11].

Fig. 17

The fraction of critical damping may be assumed in this building on the order of 10% as a minimum because of the way it is fabricated, that is to say, all structural sections are riveted together and therefore energy dissipation may be greater, mainly when the structure is forced to work with high stresses close to the elastic limit of the connections. Under limiting conditions the building can take safely base shears up to 5% of gravity.

The building was finished six months before the large earthquake July, 1957 in Mexico City and the structure and architectural details behaved very well during the earthquake throughout the building. Not a single glass window was broken. Unfortunately, no measuring instruments were installed in the building at that time and the forces of the earthquake in the building were not measured. However, from inspection of the building, it was recognized that the structural frame did work to its full capacity during that earthquake and with a base acceleration in the order of 6.5% of gravity, as obtained from recent information. It is the author's opinion that if the structure would have been reinforced concrete designed, with same shear forces the structure might have suffered some damage. This evidence proves the great qualities of structural steel construction because its large ductility.

Welded-riveted buildings

After experience was gained from Latino Americana Tower and Anahuac Building behavior during July 28, 1957 large earthquake, other steel structure buildings were designed from which two are 20-storey tall buildings. One was finished before the earthquakes May, 1962 and behaved well in all its details during this earthquake and the other constructed in 1963 passed already earthquake July, 1964 without damage. Photograph 15 shows the structural frame of the first building property of „La Comercial" Insurance Company, and photograph 16 the second property of the „Banco Internacional". These two buildings were designed with exactly the same philosophy, but with more experience concerning steel structural frame behavior, from past earthquakes. Therefore, columns in these buildings were designed as square sections welded together as shown in photographs 17 and 18. This type of section has proved to be the best for columns stressed under earthquake forces. Large I-beam sections were prefabricated with plates, angles and cover plates riveted together (19). The joints are designed of the butterfly type high moment resistant and welded to the columns as shown in the photographs. The prefabricated and standard I-beam sections were riveted in the field during erection, and in such a way as to make the rivets work under double shear to reduce the size of the joints.

In order to reduce steel and increase control rigidity of the structural frame for seismic design, the reinforced concrete floor slabs are designed to work in conjunction with the structural frame main girders by means of shear connectors. Furthermore, diagonal shear in floor slabs because of earthquake forces causing differential displacements in the structural frame are absorbed by designing diagonal steel members similar to those used in Latino Americana Tower and Anahuac Building (20).

The curtain walls of these two buildings are designed with a steel flexible frame where the curtain wall materials are attached allowing for relative displacements between floors. Interior partition walls are from the floating type, that is to say, firmly attached to the floor slab where they rest and free to slide in their plane, with respect to the structural frame. The structures thus designed are able to take earthquakes as large as that of July 28, 1957 in Mexico City, without minor damage in their structural or architectural elements. Moreover, they have reserved strength due to ductility of the structural steel frame to resist with minor damage considerable larger earthquakes that may hit Mexico City in the future.

Suspended building

The building housing the life insurance company "Monterrey" is an unusual project where the combination of reinforced concrete and structural steel was used for economy to solve the structural problem of this special design (21). The floor slabs are suspended from structural steel trusses (22), supported on large reinforced concrete beams, that form a rigid frame with two concrete piers, used to place the elevators, stairways and utilities as shown in typical floor plan (Fig. 18). The combination of materials, concrete and steel, have been used more properly for the work they are best suited, namely: compression in concrete and tension in steel. The roof floor was assigned to a restorant supported on inverted rigid frames. A suspended cable roof system supported from the upper ends of the inverted frames counteracts the effect of large bending moment effects on the cantilever arms of the inverted frame, thus introducing economy in the necessary moment resisting sections (21). The floors are of steel joists and lightweight concrete slab construction. The concrete slabs are rigidly connected to the upper cord of the steel-joists by means of shear connectors. One end of the steel-joists is connected to anchor plates embedded in the concrete shafts and to main hanging steel girders, the other end is supported by tension members constructed of two channel sections which support the six floors of the building and are held by a special designed joint resting on the steel cantilever trusses (23). The connection holding the hangers was provided, temporarily with cross channel sections, in order to be able, by means of jacks placed at each end, to adjust small differences of deflection of the cantilever trusses and strain in the hangers and to rectify the level of the floors. Later these elements were removed.

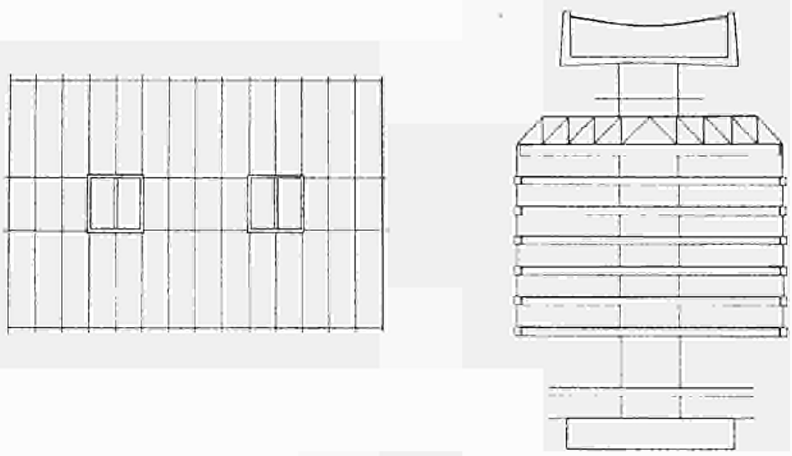

Fig. 18

The steel structure was all welded in its prefabrication phase at the shop and in its connections in the field (24). The allowable stresses in the design followed specifications of A-7 structural steel. The reinforced concrete floor slabs act like diaphragms in their place firmly connected against side-sway by the large concrete shafts supporting the framework.

The foundation was designed like a gigantic footing supporting the two reinforced concrete piers. Its area is 55% smaller in plan than the projection of the typical floors. As a consequence, the cost was smaller than a conventional foundation in Mexico City. The foundation was designed as a compensated foundation without piles and provided with a basement [13,14].

The acceleration of the ground during earthquakes transmitted to the foundation of the building, induces dynamic stresses in the structural frame and will excite the gigantic reinforced concrete frame to vibrate, with only one degree of freedom in its plane, and two degrees of freedom transversely, in spite of being a six-storey tall building. The displacement in the upper part of the building is of a small order of magnitude. The floor slabs hanging from the upper part of the building cannot experience more displacement than the concrete piers, since they are fixed to them like large washers and therefore will follow the movements of the shafts during seismic motion. The conditions mentioned before obviously make the structure mode determinate and consistent in its dynamical behavior. The complicated seismic analysis is also eliminated since the building will vibrate only in one mode, in contrast with a conventional building that would show one different mode of vibration for each floor level and during an earthquake there is always the possibility that one of these modes may be excited more than the others. The mode of vibration is on the order of 1 sec in both directions, and was checked by a sensitive instrument placed at the roof floor of the building. The building has passed already earthquakes May 11 and 19, 1962 and July 6, 1964, without even minor damage either in its structure or interiors. Photograph 25 shows the finished building.

Acknowledgements

The Latino Americana Tower was architecturally designed by Architects Manuel de la Colina and Augusto Alvarez, being Chief of the Engineers Department of Latino Americana Life Insurance Company, the brother of the author, Adolfo Zeevaert, C.E. who was encharged of the general project and construction of the building. The steel structure was prefabricated by Bethlehem Steel Corporation and erected by a local constructor of Mexico City.

The Anahuac Building was designed by Architect Juan Sordo Madaleno. The steel structure was prefabricated with Mexican steel and erected by Campos Hermanos of Mexico City.

La Comercial, Life Insurance Company building was designed by Architects Hector Mestre and Manuel de la Colina. The Banco Internacional building was designed by Architects Alberto Velasco and Javier Garcia Lascurain. The prefabrication with Mexican steel and erection of both of these steel structures was contracted to Campos Hermanos.

The hanging building property of Life Insurance Company "Monterrey" was designed by Architects Enrique de la Mora and collaborator A. Gonzalez Pozo, the general constructor was S.G. Construcciones, S.A.

The author was Consulting Engineer in foundations and structures for the buildings mentioned before and wishes to express his sincere appreciation and comprehension to the architects of these major projects during structural design and construction, to the builders, for their interest and integrity in producing a first quality work, and last but not least to the owners that deposited confidence on him to design the structure and foundation of these buildings in an environmental area of difficult subsoil conditions and earthquake problems.

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Description of photographs

1 — Collapse of upper storeys of tall building.
2 — Collapse of structure.
3 — Buckling of columns.
4 — Water tank.
5 — Failure of curtain wall.
6 — Collapse of concrete structures.
7-11 — Latino Americana Tower: steel erection finished — foundation structure — structural frame during erection — shear connectors — finished building.
12-14 — Pictures of Anahuac Building — two steel structure views.
15 — Structural frame "La Comercial".
16 — Structural frame "Banco Internacional".
17 — Welded steel column, custom section.
18 — Structural frame "La Comercial" — composite riveted.
19 — Composite riveted beams "Banco Internacional".
20 — Diagonal steel reinforcement.
21-25 — Structural frame "Monterrey" : general view — detail — concrete piers — steel trusses and hangers — picture of finished building.


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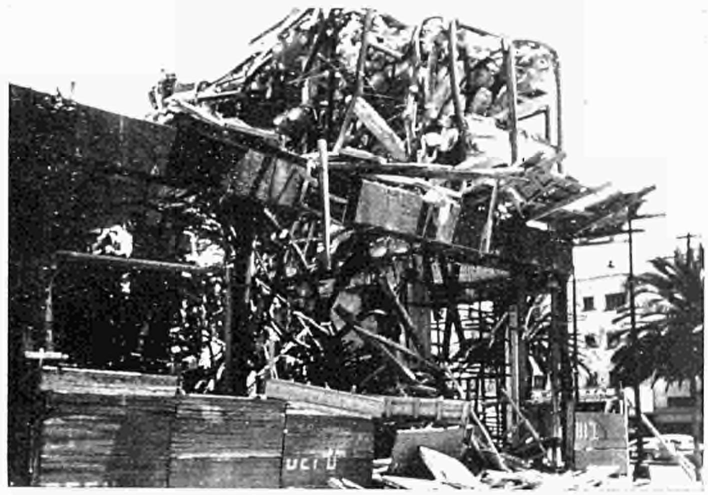
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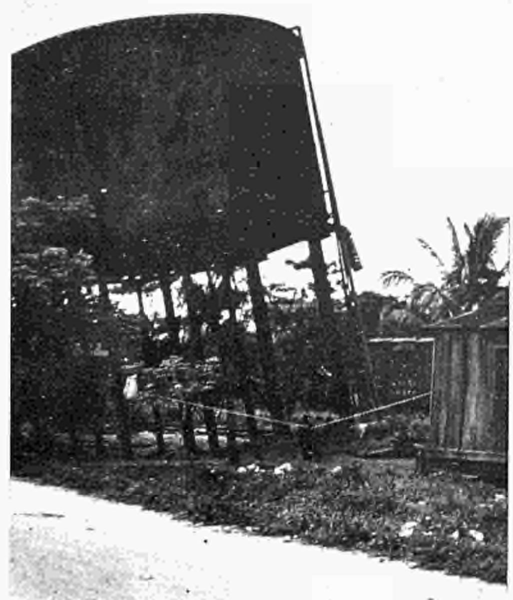
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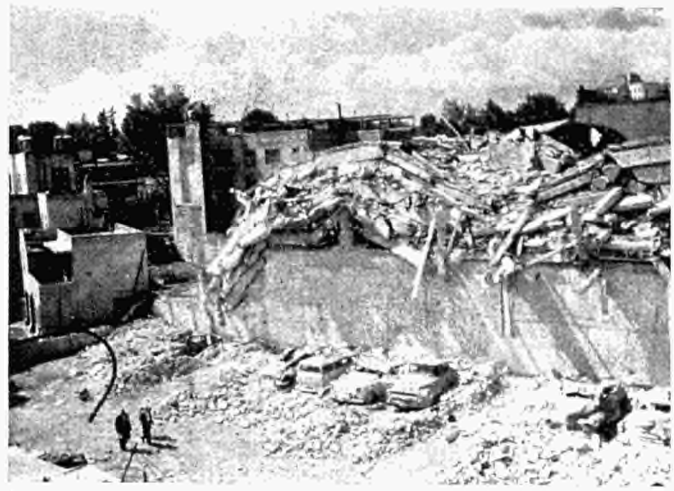
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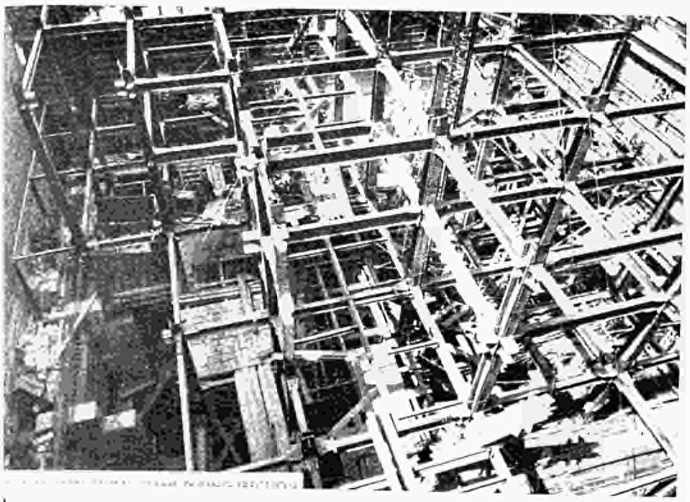

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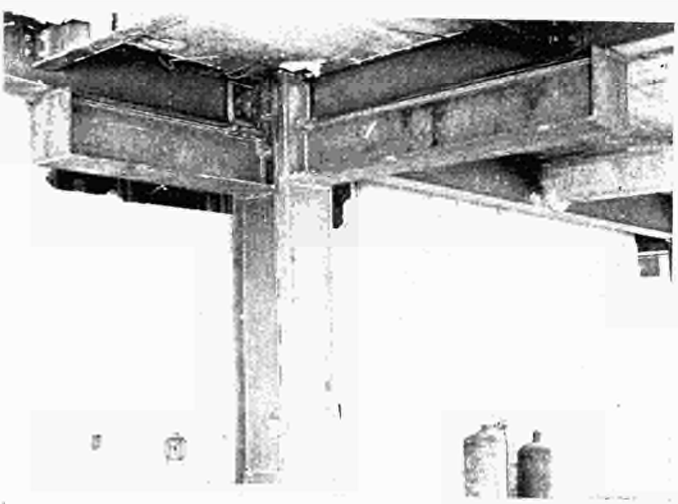

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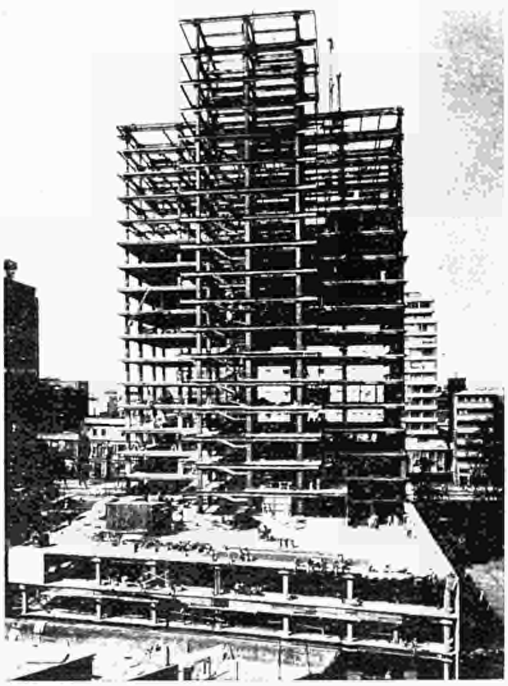

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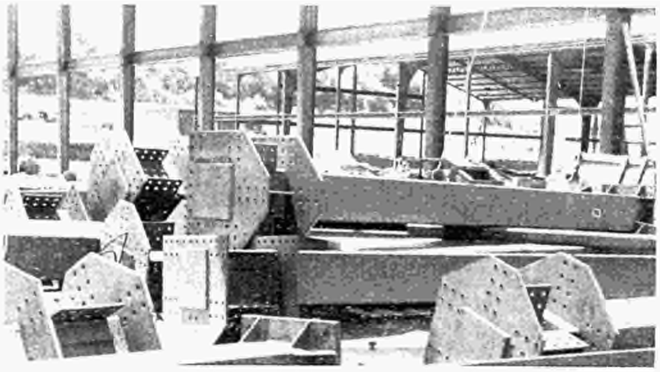

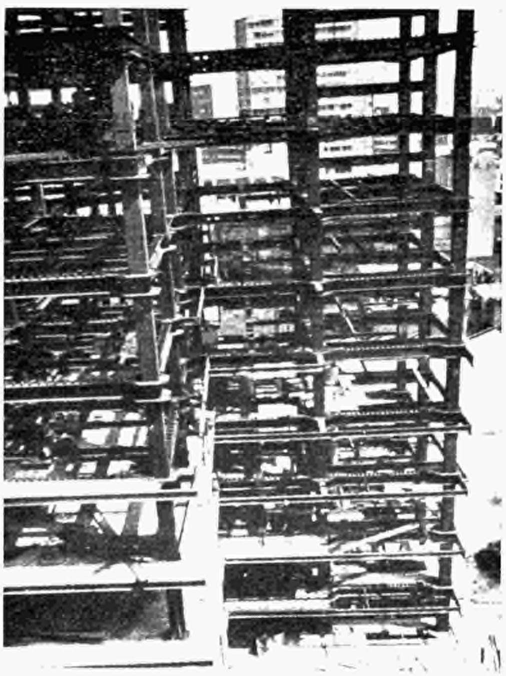

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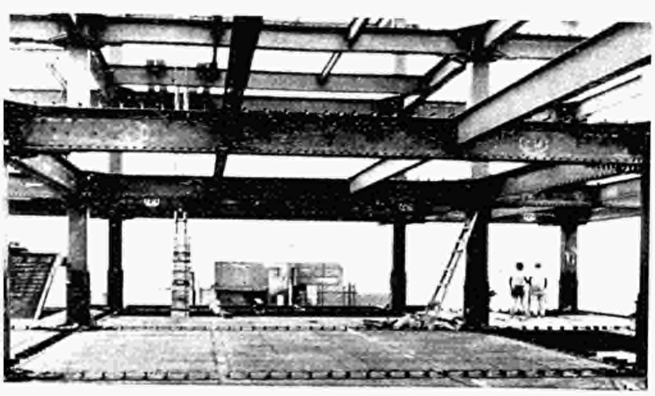

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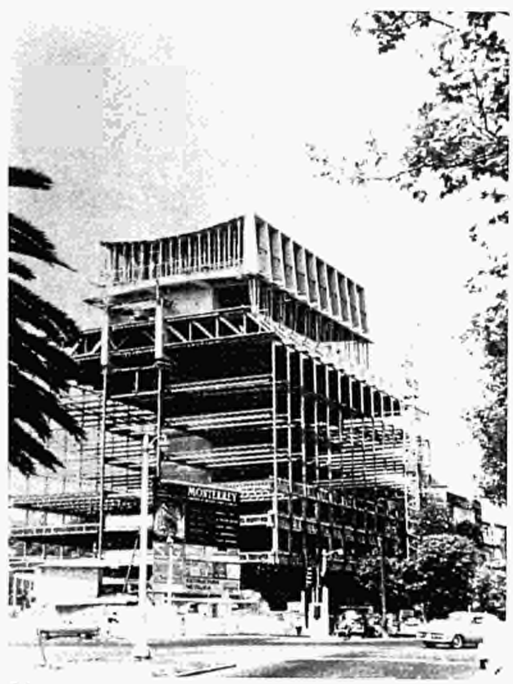

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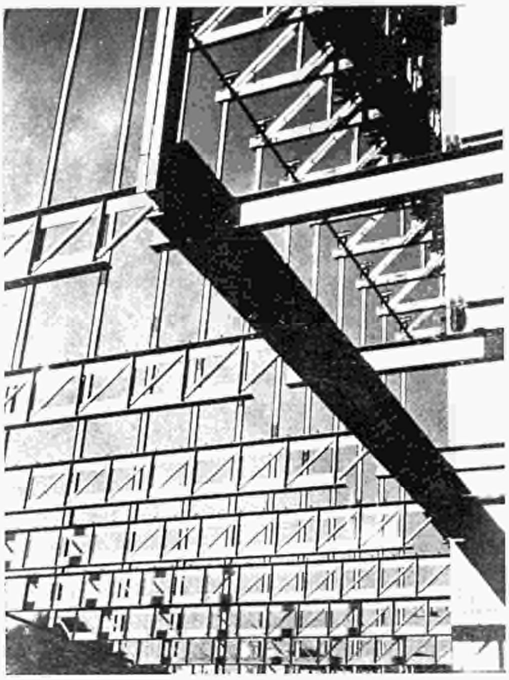
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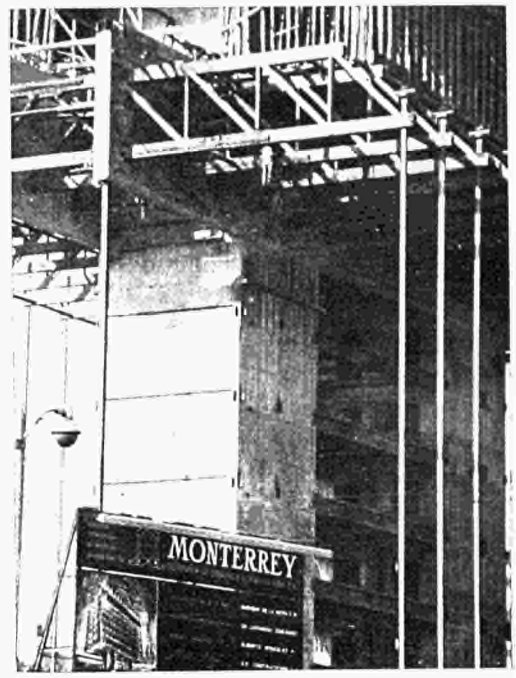

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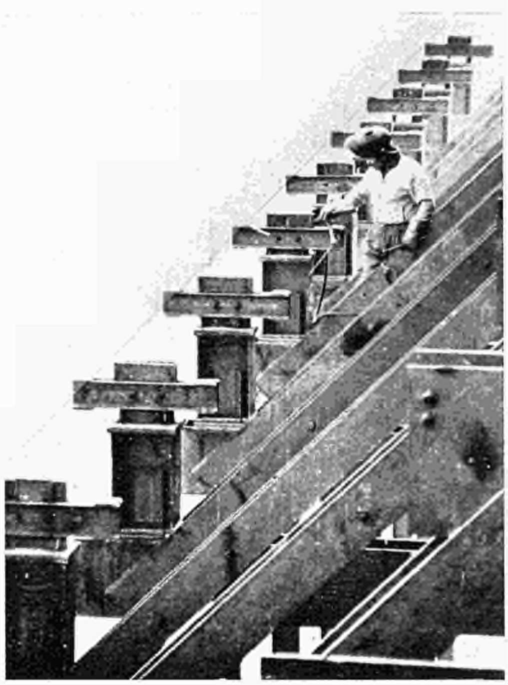
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Curt F. KOLLBRUNNER

Present and Future Structural Steel Framework

(Original text: German)

General

The term "steel-framed building" covers in its narrower sense multi-storey buildings and in its wider sense all shed-type buildings with a steel framework. We shall, therefore, here consider both types of building.

Steel is used in building work, first of all on account of its high and consistent strength and the consequent small sectional sizes of the structural elements, and secondly because of the rapid erection which requires little working space, and also on account of any possible later additions which can be carried out with relative simplicity. With steel, one achieves the largest possible usable space and, where required, column-free rooms and halls. The comparatively light steel framework with light partitions weighs considerably less than a concrete — and — masonry building. With the self-weight being smaller, the foundations are more economical.

The aim of modern engineers and steel fabricators is today to build as light as possible; avoiding large expenditure of effort on work in the fabricating shop. The constant increase in wage rates must be offset by better work organization; by rationalization of work in planning, fabrication and erection; by extensive standardization; and by constant testing of fabricating methods. That is to say, new steel structures will be developed in such a way that with small specific steel consumption wage costs will be kept down to a minimum by rationalized fabricating processes. However it should be remembered that these rationalized fabricating processes necessitate a large capital outlay in the fabricating shop.

Until a few years ago, there were only two methods of production for steel-framed buildings: one built either heavy, — that is with a high overall price of the material and low fabricating costs, or one built light, that is with the material cost low but with considerably higher fabricating costs. For each individual case it had to be established which was the more economical way: to use more steel, that is to use heavy sections in order to save on labour costs, or to allow for a larger wages bill for the fabrication and to save on the steel itself.

Today we have advanced considerably. Thanks to the most up-to-date practical knowledge, experience and research, to rationalization and to the adaption of fabricating shops to the latest techniques, one can now combine the advantages of both methods in designing, constructing and erecting structures, not only light in weight but also cheap to produce. At the same time, this lightweight construction should not be carried too far — we are designing buildings, not aeroplanes!

In Switzerland welding and high-strength friction grip bolting have replaced rivetting. Welding not only produces a saving in material; it gives a direct and undisturbed flow of the internal forces, which corresponds to an increase in quality. In addition, a welded structure is more aesthetic.

Assembly joints are welded or bolted. Where high-strength bolts are used, surface transference of load takes place with an improved stress flow as compared with ordinary bolts, which produce small stress peaks. The upper limit for span is, and will still be governed by steel, despite competition from other materials, as for example pre-stressed concrete; high-grade steel is often used for such structures.

Sheet steel, in the form of bent thin-wall sections, is finding increasing use in steel-framed buildings. In the calculation of these thin-walled sections the stability theory, the torsion theory and the folded plate theory are much used. It should be noted that stability theory calculations, proved by numerous experiments, utilize the plastic as well as the elastic range [1].

In order to publish, whenever possible, uniform European Standards, Committee 8 (Stability Problems) of the European Convention of Constructional Steelwork Associations has carried out further tests. For the past four years the TKSSV (the Technical Committee of the Swiss Steelwork Association) has been concerned with the moden torsion theory. The method of calculation has been laid out in several papers [2] A fundamental paper, which describes the relationship between the torsion theory and the folded plate theory, goes to press at the end of this year. With its publication we shall have all the worthwile rudiments for the satisfactory and rapid design of light steel buildings: we can save weight and design economically.

In addition to these theoretical design matters, we must in future make sure that as far as possible, mass-production lines can be set up in our fabricating shops, on an automatic or semi-automatic basis. This is of course, often very difficult to achieve, since steel-frame building in Europe is on the whole still to be compared to made-to-measure and not ready-made tailoring.

What we need today is a still better knowledge of the material's working properties, for only with this knowledge can we produce modern structures and achieve any success.

Basic, as well as applied research must be constantly pursued not only in the technical colleges but also by the Constructional Steelwork Associations and by individual firms. Only through calculating intellect and formative feeling does one come to creative achievement. Close co-operation between engineer and architect is ever more necessary and this must of necessity begin at the design stage.

Advantages of steel in building work

At the first Swiss Steelwork Congress in 1953 in Zurich, I listed twenty outstanding advantages of steel as a structural material and I will not repeat them here [3].

Steel is, and will remain, the ideal structural material for tall buildings, where the applied leads are considerable and the spans range from medium to large.

Whilst the concrete people make full use of the term „prefabricated structures", it should be remembered that all steel buildings are of prefabricated construction. The concrete structure has merely copied the steel frame, and the erection of the concrete structure is certainly more complicated than the erection of the steel one.

Every forward-looking, creative engineer aims at greater economy with adequate safety [4]. Economy can then be achieved by the higher strength of the steel as well as by better detailing. In both spheres, in the strength increase as well as the construction details, much progress has been made in recent years; nevertheless we are only now at the beginning of a new epoch and much more research still needs to be done.

Multi-storey buildings

Multi-storey buildings, in which the load-bearing framework is of steel, are put up specifically a industrial buildings, office blocks and flats. With an all-steel skeletal structure the columns, beams and frame are of steel. Because the partitions and floors are independent of the steel frame, the space requirements of the building can at any time be adjusted.

The constantly changing organization has to be considered in industrial and office buildings. The greater the flexibility of such a building, the better can it be adapted to the ever changing requirements of the administrative structure. This leads to widely spaced, often internally column-free frames, in which the internal walls can be placed optionally and later rearranged at will. This means that the steel firms tender not only for the steel frame itself but also for the main partitions and supply and erect them together with the steelwork.

Figures 1 and 2 show outline drawings of multi-storey buildings. The erection methods shown in Fig. 2 are exceptional to the normal run ; generally erection proceeds from the bottom to the top.

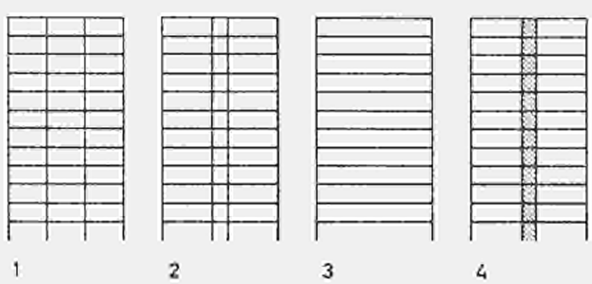

Fig. 1 — Multi-storey Buildings. Details
1. Regular arrangement of the columns.
2. Irregular arrangement of the columns.
3. No internal columns.
4. No internal. columns. Concrete core.

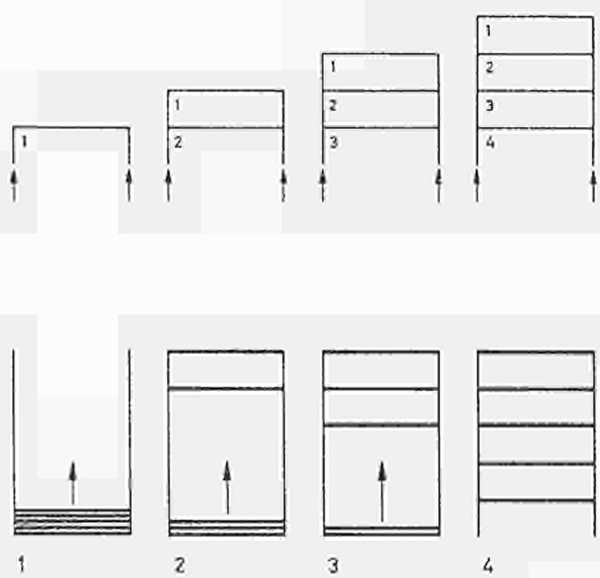

Fig. 2 — Multi-storey Buildings. Erection, details
Upper : Erection with jacks (as in boiler erection)
Lower : Concreting of the floors at ground level and subsequent jacking up.

In future, for certain buildings, it will be likely that the constructional steelwork company will be the general contractor and will supply the whole building, inclusive of foundations, to the client ready for occupation,

Shed-type buildings

Shed-type buildings consist mainly of only one storey, *i.e.* usually of roof and enclosing walls. They cover all intermediate stages from the steel roof on concrete or masonry load-bearing walls to a complete steel portal frame, which comprises the load-carrying parts of roof and wall, together with all cladding and bracing. Here the fine lattice work of a few decades ago has been supplanted by solid-web girders and triangular trusses with graduated member sections and simple panel point details.

Fire protection

For over five years we have known that modern steel-framed buildings need not fear fire. We proved then by extension tests that a fire loading of 25 kg/m² is not dangerous for steel (5). For a long time we have been insisting that, for fire loadings up to 20 kg/m², steel need no longer be clad [6].

The fire loading of a building or of part of a building comprises the calorific value of all combustible material present in the part of the building under consideration, based on the unit of floor area. Converted into a corresponding quantity of timber, per unit of floor area, the fire loading is expressed in kg. of timber/m² of floor area.

Because the present-day fire loading of buildings is, in general, only 8 to 15 kg/m², the steel frame need not be clad.

Unfortunately, the principle of the fire loading being the main criterion of the fire hazard is today still opposed by people not fully acquainted with the subject. They fear that the fire loading could be greater than that originally intended owing to a later, changed use of the building. This false argument can be countered by what already happens in practice, namely the application of building statics, in which the members are not designed for static and dynamic overloading, but only for the given loads. Consequently, the exaggerated fire protection measures and cladding of steel structures, where the fire loading does not exceed 20 kg/m², must be discontinued.

The fire tests are performed in most countries under similar conditions. However a clear dividing line must be drawn between the standard curves and actual fires. A fire develops not according to a standard curve, but according to the existing conditions: exposed to a current of air it develops quickly, and with little wind it develops slowly and smoulders.

In a natural fire the actual temperatures are in general much lower than those indicated by the Norm or Standard Curve (Fig. 3). In reality the Temperature-Time Curve develops generally in such a way that after the start of the fire there is a rapid increase in temperature to a peak lying above the standard curve, after which the temperature gradually decreases.

Where the fire loading is greater than 20 kg/m² the steel can, according to present-day practical knowledge, be clad much more simply than is prescribed by the official fire regulations. For example, for columns of wide-flanged or similar sections, it is sufficient that these be concreted between the flanges, leaving the flanges themselves unclad.

Although our experience is not complete, a coat of protective paint will probably suffice in the future for cladding the steel structure; in a fire the coat turns into a protective foam.

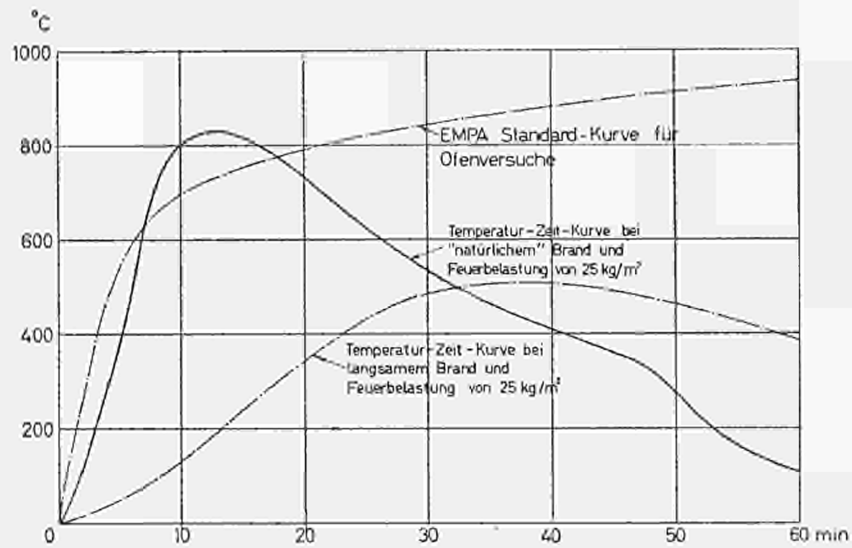

Fig. 3

Protection against corrosion

The protection of steel structures against corrosion no longer poses any problems. Given careful sand-blast cleaning with subsequent appropriate base and surface coats of paint, the maintenance costs are no greater than for reinforced concrete buildings. It should be noted that the cold zinc process has proved very effective in building work.

Retrospect and prospect

The earlier distinctly unenterprising approach of architects and engineers who designed with the more familiar concrete rather than with steel, is fast disappearing. Percentagewise, more and more steel buildings are being erected, in which the steel is deliberately emphasised and no longer concealed as a structural element. The light, pleasing, airy, simplicity of the steel frame building comes to the fore in its façade.

Modern façade units have been improved to such a degree that they will last for decades without any large degree of maintenance. These provide insulation against heat and cold, and have a very small self-weight.

Wall elements, as partitions, have also been much improved; the same cannot be said of ceiling slabs.

The ideal slab should be capable of supporting its load over large spans, be of light weight, be able to act as insulation against noise, and as far as possible be produced dry.

The last ten years have seen a transformation in the outward appearance of buildings. Though formerly the steel frame was hidden behind inappropriate cladding, nowadays it is allowed to show; *i.e.* it is made use of as an architectural element.

It should be borne in mind that today steel buildings play a much greater role than steel bridges. Prestressed concrete has won much ground in connection with short and medium spans, while in building work the development is in the opposite direction. Modern architects know that they can build aesthetically, practically, rapidly and economically with steel. The steel frame and the often metallic façade elements complement each other. Both are space-saving constructions and both possess similar accuracy of manufacture.

In building work there is no "war to the knife" for the progressive engineers with their different building methods. Everybody knows that only he can win who submits the best design. However, everyone also knows that the best design can often be created only by a co-operative effort — steel and reinforced concrete; consequently one often finds a combination of the rival methods.

The steel frame does not always have to carry the full loads. The steel frame can be designed to carry only the vertical loads; the horizontal forces can, for example, be taken by a concrete core (stair-well, lift shaft).

More stainless steel and aluminium alloys will be used in the framework, when the costs are lower.

In addition, the all-too-large window areas, which are generally covered by curtains in the summer anyway, will probably disappear.

Whereas those parts of the load-bearing structure lying under the ground used to be designed generally in reinforced concrete, the tendency nowadays is to make the whole frame, including that part underground, of steel in order to obtain a degree of unity. This also saves building time.

It must not be overlooked that the increase in unit cost over the last few years has been considerably greater in the building industry than in the steel trade. This is another reason why steel-frame buildings are becoming increasingly competitive vis-à-vis concrete structures. That steel-framed buildings give better protection against earthquakes is merely noted in passing.

The future is rosy for steel-framed buildings. We have clear, straigthforward methods for calculation, fear neither fire nor corrosion, can erect the buildings more quickly than hitherto, thanks to rationalisation in the office, the fabrication shop and on the site, and can give the client an economical building that he can at any time adapt to changing requirements. Steel guarantees, not only now but also in the future: in fact light structures providing much useful space; structures which meet all architectural requirements, and which, without further large costs, can at any time be adapted, changed, heightened or widened as the case may be, to suit the latest circumstances and conditions.

The intelligent, forward-looking client knows today that if he builds in steel, he will be in good hands, provided the constructor appreciates the inter-relationship between research and economic steelwork, takes this into account, and builds better than in the preceding years.

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Description of photographs *

1-3 — Extension of an Assembly shop of AG Conrad Zschokke, Döttingen; two pinned frame. Steel frame unclad — two pinned frame. Steel frame unclad.

3 — Machine shop and Warehouse, large window frontage, steel frame unclad.

4-5 — AG Brown, Boveri & Co., Birrfeld: Shed of prefabricated and prestressed concrete. Steel frame unclad. — Internal view. Steel frame unclad.

6 — Huber, Pfäffikon; erection photograph, no internal columns.

7 — Sulzer Brothers, Solothurn; detail photograph of a panel point.

8-12 — Cantonal School, Baden: erection photograph — erection photograph — elevation — internal photograph, steel frame partly unclad, internal stanchions partly clad with concrete. Aerial view. In general, in all buildings the steel stanchions and the underside of the steel beams remain visible.

13 — Hürliman Brewery, Zurich; bottle factory, erection photograph.

14-15 — Haus Waltisbühl, Bahnhofstr., Zurich; photo by day — photo by night.

16 — SAFFA — Tower House, Zurich; unclad steel frame.

17 — Beauregard Brewery AG, Freiburg; unclad steel stanchions in façade.

18-19 — Caisse Centrale d'Allocations Familiales, Paris; administration building. Only relatively norrow strips are glazed. Movable partitions allow optional room arrangement — erection photo, multi-storey frame with cantilevered cross members.

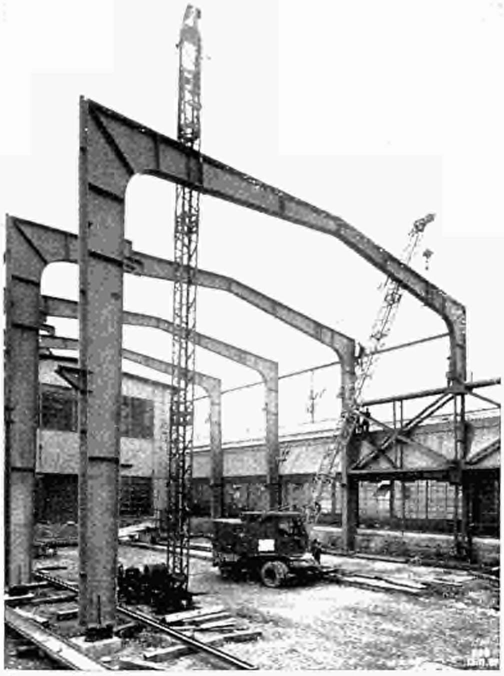
20 — Noell & Co., Würzburg; administration building.

21 — Mannesmann AG, Düsseldorf; administration building. The steel frame is designed for vertical loads only. The horizontal forces are taken by a concrete core (lift shaft, stair well, etc.) which is fixed in the three-floor foundation block.

22 — First National Bank, Minneapolis, U.S.A.; business building. Prefabricated, continuously glazed aluminium façade.

23 — Sociedad Española de Automoviles de Turismo, Barcelona; warehouse and display-building. Main façade fully glazed.

* Photographs 17-25 are taken from the book : Bauen in Stahl 2. Published by Swiss Constructional Steelwork Association, Zurich, 1962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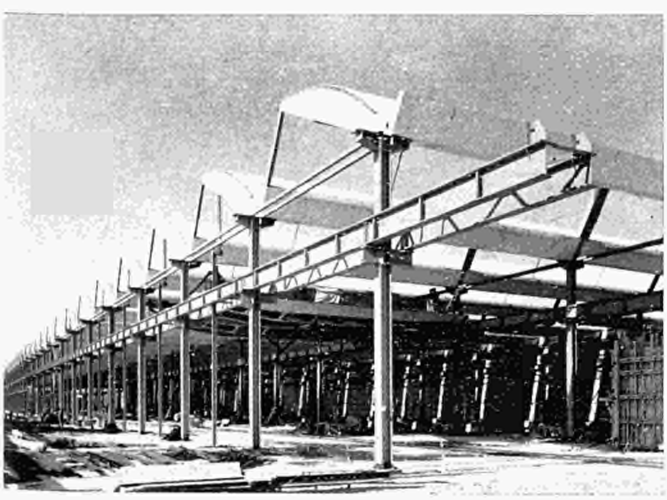


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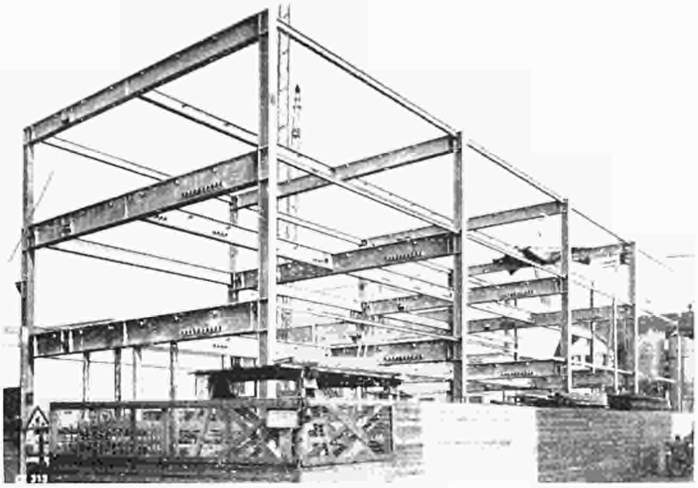
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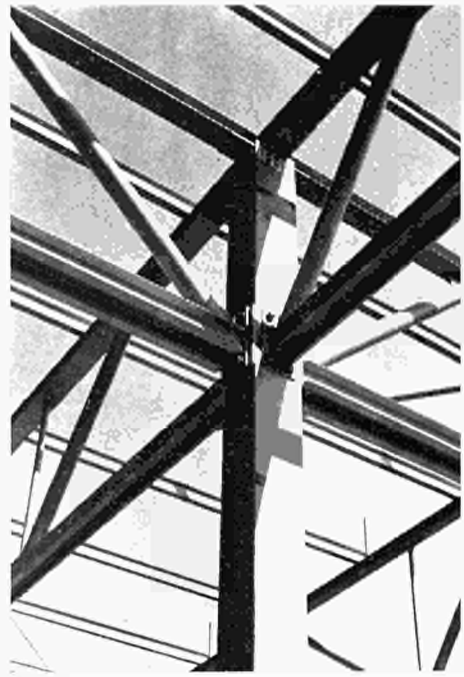
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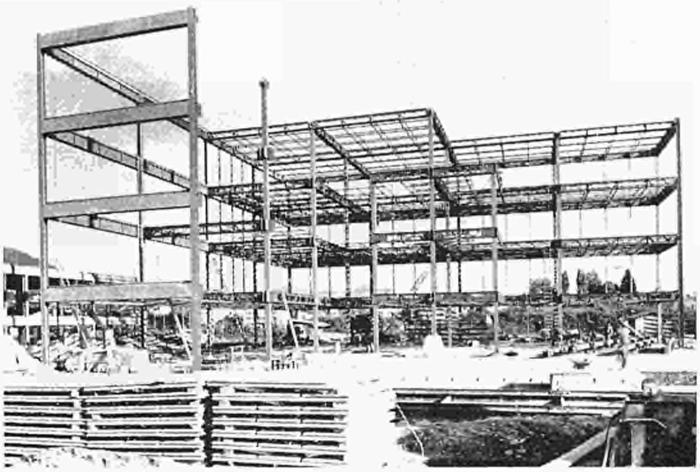
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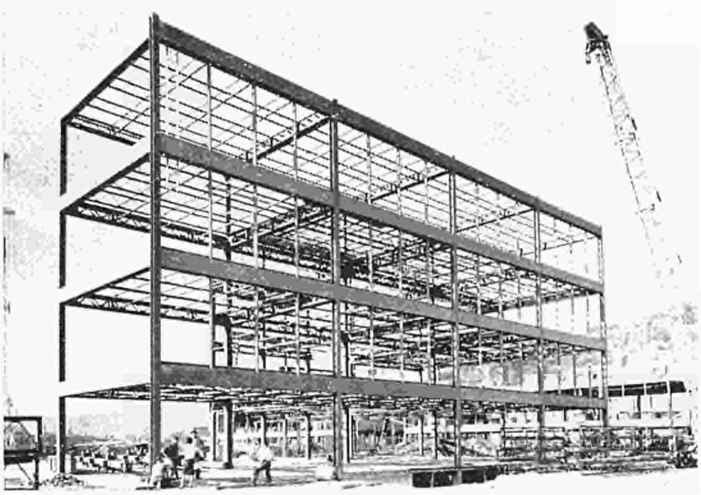
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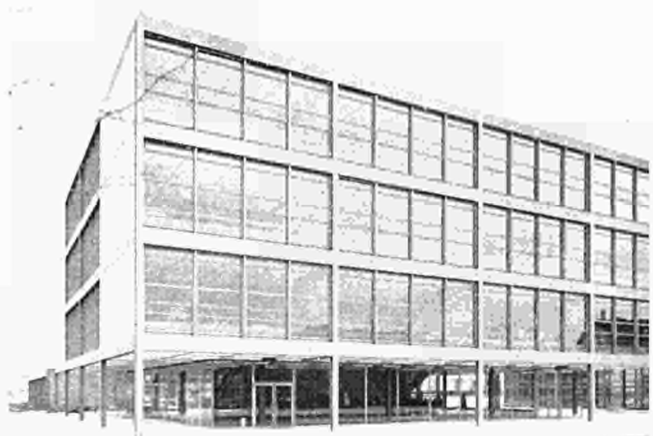
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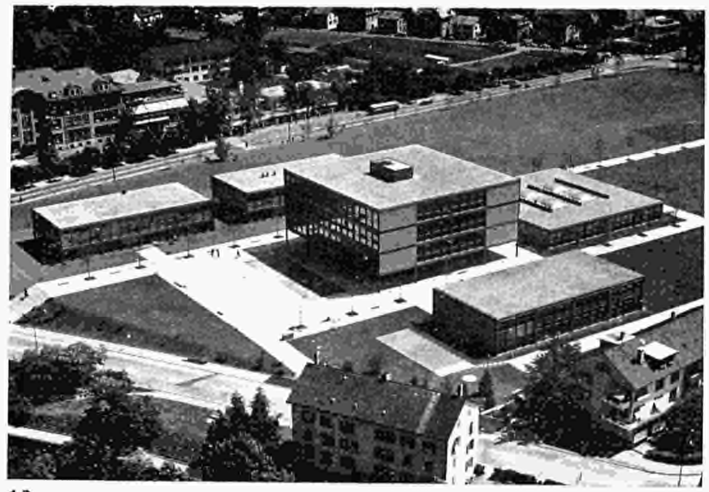

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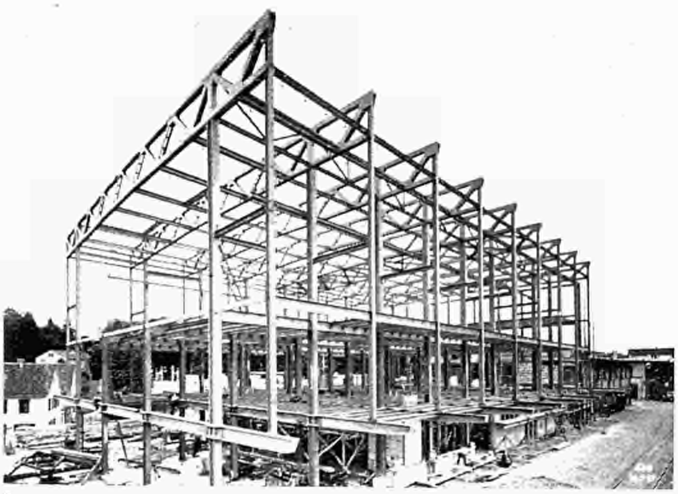

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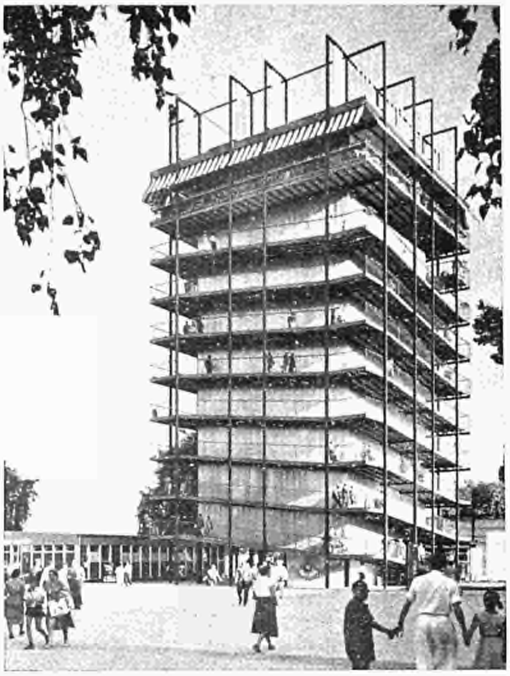

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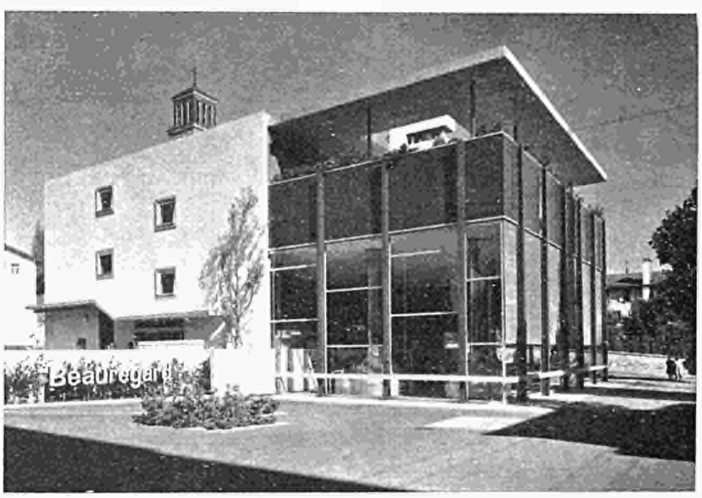

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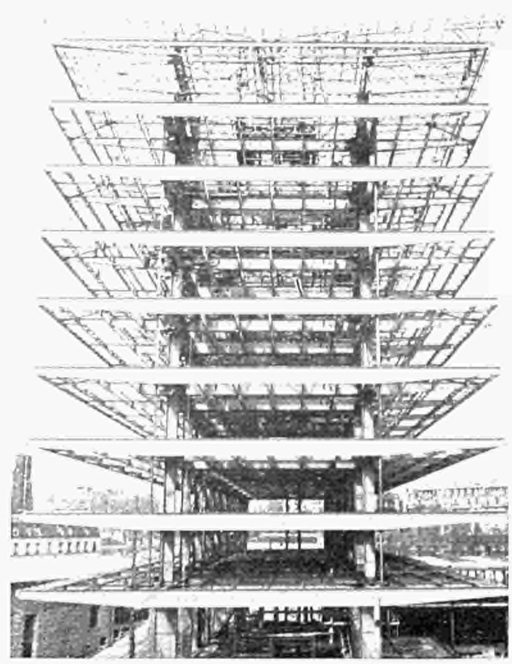

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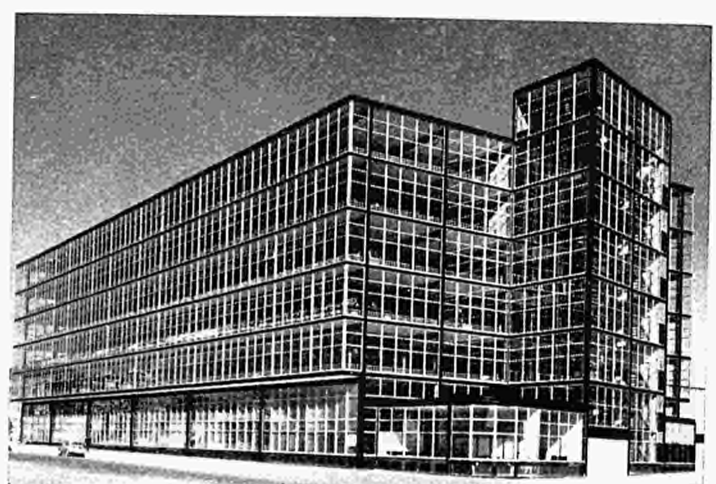

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Giacomo SPOTTI

Lightweight Structures in Industrial Buildings

(Original text: Italian)

Introduction

It would seem difficult to find anything fresh to say on this subject after the detailed proceedings of the XXIst International Congress of Steel Information Centres at Harrogate, England, the reports to the Graz Congress on Austrian Steelwork Constructions in 1963 and the wide field covered by the Conference on Italian Steelwork Constructions held in September 1964, not to mention the comprehensive literature which has appeared both in the E.C.S.C. countries and elsewhere in Europe.

In an attempt to crystallize the situation, however, I think it may still be worthwhile stressing one or two technical and economic aspects suggested by twenty years of personal experience in constructional steelwork design and engineering in Italy.

I have purposely refrained from providing photographic or diagrammatic examples in order to ensure fluency and consistency in dealing with the various aspects of the subject.

The bibliographical references quoted in the paper and listed at the end will, I trust, fill this gap, especially as they emanate from experts far better qualified than myself to handle this question.

Planar grid frameworks

As lightweight constructions, the steel grid structures of the closing decades of the last century and the early part of our own were characterized by the mechanical repetition of patterns which until then had been produced entirely in timber, perhaps using iron tie bolts.

This was the period of triangular roof trusses and lattice girders consisting of conventional rolled sections — a far from lightweight structure despite the static and compositional nature of the designs involved.

The subsequent appearance of seamless tubular sections has greatly differentiated lightweight structures not only from solid web constructions but also from the lattice system previously adopted.

Designers and engineers appreciate the greater compressive flexural strength and resistance to fatigue of the circular section, combined with its improved mechanical properties. Its appearance has won the favour of architects seeking modernistic effects, and clients have been attracted by the possibility of saving weight, even if this has not always been accompanied by a saving in costs.

Certainly as regards Italy it has only been since the last war that interest has begun to revive in open shapes and tubular sections, no longer in the form of repetitions of conventional patterns based on the use of outmoded materials but, cautiously at first and then with increasing confidence, in static designs of either complete assemblies or structural components better suited to being produced in steel and more capable of profiting by its particular properties.

Bracing appears much less frequently in frameworks and there has been an increasing reliance on hyper-statical structures. The mesh of the grid has tended to expand while the dimensions and shapes of the individual members are such as to provide the required strength, rigidity and stability.

For the basic members such as main beams, joists, purlins and cross-ties — either open rolled sections, tubes, or round bars, use only — or else, (just as frequently,) sometimes successful combinations of two of these types such as tee sections, angle ties, and bar and tubular wall-support members.

Structures which have already won recognition in the U.S.A. are now spreading to Europe, where labour costs are lower and working hours longer. An example is the use of open-web steel joists.

The 12th edition of the important German publication *Stahl im Hochbau* provides, under the title of *Leichtbinderarten* (lightweight framing methods), a further detailed account of the types of construction which are gaining ground in Central Europe.

Generally speaking however, whereas the use of mechanized production plant for standardized lightweight trusses has become a necessity in countries such as the U.S.A. with high labour costs, the economic advantage of this type of structure in Europe has become less marked owing to strong competition from other forms of steel construction.

This will certainly be the case until designers, architects and clients adopt a more progressive attitude towards modular systems, so as to provide an incentive for investing large sums in mass fabrication, *i.e.* until a greater awareness of the possibilities of steel leads to a wider acceptance of steel constructions provided under the stimulus of progressive trends of thought.

Adverse tendencies in fabrication

Scope for criticism exists, however, in the predominantly, if not exclusively, commercial tendencies exhibited by those organizations inspired by sales rather than methods, and still less by design considerations, which have sprung up in various parts of Italy in recent years. Firms of this kind have overcome rising labour costs by resorting to independent groups of the small jobbing kind whose facilities and, to an even greater extent, technical knowledge are often elementary or deficient.

Although activities of this kind have often been temporarily successful financially, thus inviting credence being given to the total viability of the system, frequent technical failures have combined to prevent the widespread recognition of lightweight steel constructions among the Italian clients.

Improvisation in methods of fabrication has often gone hand in hand with even greater fallibility in selecting the materials, metallurgically most suitable.

Inadequate testing and the lack of technical bodies responsible for approving products on the basis of appropriate standards has been a further factor in encouraging undesirable features such as a partial or total lack of suitable methods of fabrication, particularly in the case of many of the smaller industrial buildings.

Design standards and methods

This regrettable lack of standards is only offset by the Standard Specifications and Load Tables published as a result of careful static tests and thoroughgoing investigations by the U.S. Steel Joist Institute, set up in 1928 by American constructors.

An even greater problem arises, now that sections which are cold-rolled or pressed from steel strip are available to firms of the type mentioned above. These offer attractive possibilities of use but are subject to the drawbacks of improvised fabrication and inadequate technical knowledge.

It is obvious that traditional structural calculation methods and existing European national standards can only be relied on to a limited extent in designing lightweight structures using light-gauge cold-formed shapes produced from steel sheet or strip.

The minimum thickness permitted by existing standards and the maximum slenderness ratio are a serious obstacle to the development of this very individual type of lightweight structure. Lightweight constructions cannot and must not be dangerously synonymous with a lower safety factor.

Cold-formed shapes and cold-rolled sections offer the following advantages :

(a) they extend the working range of lightweight structures, normally used for large spans and modest loads, to medium spans;

(b) they ensure substantial if not outstanding dimensional stability and rigidity;

(c) they open the field to an extremely wide and theoretically unlimited variety of shapes, thus encouraging creative architectural expression in the design of new profiles for members and in the freedom with which standardized components can be used in both conventional assemblies and new patterns and effects;

(d) they accentuate the modern trend of architectural design towards "dematerialization", which above all is capable of development by the use of space-frame structures.

It is therefore evident that a reduction in the quantity of steel used must be achieved by improved utilization of material and rational design of complete structures and individual components.

This is the essential point about lightweight constructions, to which so much has been contributed by aerodynamics, and it is the only valid basis on which they can win permanent recognition.

Compressive flexural strength, torsional-flexural strength and fatigue-strength in lightweight sections, the concept of effective cross-section and optimized width-to-thickness ratios — all these, along with a thorough understanding of metallurgical properties, must become the everyday language of designers, architects, structural engineers and builders.

The process of familiarization must not stop at a mere appreciation of some mechanical properties of cold-shaped sections such as yield point and tensile strength, however important these may be, particularly as regards static functions. Valuable though these are, they are not the only qualities involved in the life of a lightweight steel structure, nor are they such as to preclude a fuller and more detailed investigation of the metallurgy of cold-rolled shapes.

Familiarization of this kind must lead to the development and selection of more suitable fabricating, assembly and protective processes, in addition to the adoption of more rational and purposeful designs, both static and architectural, for individual components and complete structures.

The increasing volume of research by expertly staffed and well equipped research centres and design institutes in European countries, taken in conjunction with the Instructions issued by the American Iron and Steel Institute, the German DIN standards and the detailed literature which has appeared in recent years, prompts us to examine the technical and economic aspects not only of this specialized field of lightweight construction in particular but also of the other structures referred to, with a sense of responsibility and guided by uniform criteria and standards.

Designers and constructors therefore look to Working Party 8 of the Convention of European Constructional Steelwork Associations for a vigorous and permanent contribution to the development of lightweight steel structures.

Space frames

Before closing this survey of structural developments, it is worthwhile devoting a moment to space structures.

Extremely light and slender members such as tubes, cold-rolled sections, round bars and steel cables, particularly the latter, constitute the basic elements of ultra-lightweight steel structures of the three-dimensional or shell type, which can be broken down into grid elements.

Nature provides architecture with many examples of structural dimensions varying in pattern and effect — the conch shell, the sea-urchin, the tortoise, the egg, the walnut, the mushroom and serving to create a universal and expressive language.

In the treatment of forces, in their axial distribution and in the reduction of stressing moments, special attention must be paid to even the slightest eccentricity in the application of axial loads and, especially as regards light-gauge tubes, to the least amount of non-uniformity.

The round steel bar has clearly met with widespread success in the field of three-dimensional structures than the flat beams.

Welded lightweight steel frames have become standardized for ground floors, intervening storeys and roofs and have enabled a great variety of triangular or rectangular-section wall-support members with a rectilinear or curved axis to be adopted.

Techniques, systems and methods of calculation for lightweight structures with light-gauge components will become used more increasingly as the ultra-light-weight hot-rolled sections now entering the field of steel production make a bid for recognition on the basis of high-strength properties combined with a continual reduction in weight and thickness. The probability exists that the IPE series of beams recently introduced on the European market will be superseded by an even lighter series in the not too distant future.

It is much more necessary, rather than merely desirable, that identical elements be used in lightweight flat and space-frame construction. This development would reconcile lightweight construction with the needs of mass production and favour the adoption of modern fabricating methods.

Cost of materials

So far we have concentrated on the fabrication aspects of lightweight constructional design, but we should also bear in mind the cost of materials, particularly tubular, cold-rolled and bent sections, which are the types of members most capable of meeting the complex strength and stability requirements involved on both a local and overall level.

The unit cost of these items is one-and-a-half to two times the equivalent cost of hot-rolled sections, which, on the other hand, do not usually offer the same quality and degree of strength and stability properties as are acknowledged to be possessed by tubular or box shapes of the simple shell or composite type.

Designers and fabricators of lightweight steel frames need to concentrate, rather than having a wider choice of sections, on the existing range of sections being available at prices which can compete with the so-called heavy solid girders or I-sections and, to an even greater extent, with non-ferrous and non-metallic materials.

We must be constantly aware of and stimulated by the fact that competitiveness is not limited to the load-bearing structures alone but extends markedly to attachments and covering, roof cladding and flooring materials, both panel-type and framed, which often represent the most expensive factor in building costs, even in the case of industrial premises.

This is a very wide field in which not only technicians and experts are required at all levels, but also a considerable investment in the mass-production of load-bearing members and components required for completing the buildings. A valuable example of this is provided by the German plant at Raum-Hamm (see *Der Stahlbau* 5, 1964, p. 138).

Protection against corrosion

It will be instructive to conclude with a reference to the much discussed question of protection against corrosion. Although its importance extends beyond the field of lightweight structures, it is nevertheless true that the latter specifically require this protection because the lightness of their gauge offers no margin of thickness. From this point of view round bars are preferable because of their smaller surface area and greater concentration of material in individual members.

Corrosion protection based on anti-rust paints is not always sufficiently long-lasting, not because inadequately protective materials are employed, but because methods of preparation and application are unsuitable, also the protective material is not the right one for the atmosphere concerned.

The basic reason for errors of this kind is to be found in the use of labour which is inadequately trained and far from appreciative of the importance of corrosion protection.

The need for mechanized methods of manufacturing and applying paint is especially acute at the stage which these steel frames have now reached if systematic quality levels, and economic advantages are to be safeguarded.

It should be noted that considerable advances have been made in the construction and use of electrostatic painting equipment and in the development of suitably conductive paints and additives.

In preparing light-gauge members for spraying, particular care must be taken not to cause deformations due to impact load or the thermal effect of sand-blasting. From this point of view the mechanical-jet sanding machines now on the market promise better results than the compressed-air type.

Another interesting point to note is that steel structures erected at high altitudes require much less corrosion protection than those in low-lying areas.

A more thorough examination is required of the metallurgical aspects of dip galvanizing. The spraying method, which is used to advantage in other kinds of structural steelwork, is not employed for lightweight constructions, precisely because of the latter's increasing trend towards dematerialization.

Hot-dip galvanizing is not necessarily the answer, but it needs to be considered mainly in order to demonstrate the complex differences of opinion on the subject, and thus the need for E.C.S.C. to provide a convincing answer for designers, builders and clients.

The research results reported at the Sixth International Conference on Hot-Dip Galvanizing (Interlaken, June 1961) do not seem to be borne out by the findings of the German research workers in their subsequent and most recent investigations.

Metallurgically speaking, the more important points concern pressure-welded composite forms, cold-rolled sections and the chemical composition of the steels to be galvanized, especially as regards their carbon and silicon content.

Agreement does exist, however, on recommendations for preventing deformations due to sectional asymmetry, and for preparing and operating dipping baths and constructing galvanizing plants.

Conclusions

The subjects touched on during my brief historical survey of the development of lightweight steel structures reveal the following :

(a) the variety of overall patterns and individual members in lightweight construction ;

(b) the adaptation of standardized forms, *i.e.* of mass-produced components, to the numerous types of architectural design ;

(c) the consequent possibility of a successful compromise between the needs of a creative language and the dictates of economic competition ;

(d) the urgent need for large-scale visual and documentary promotion in order to develop an awareness of steel at all building-industry and client levels ;

(e) the advisability of architects, designers, structural experts and builders acquiring a more thorough knowledge of the metallurgy of steel sections ;

(f) the necessity to introduce specific standards for lightweight steel structures in the E.C.S.C. countries quickly and on a uniform basis—in the absence of European standards, reference should be made to the "Specification for the design of light-gauge cold-formed steel structural members", possibly with additions ;

(g) the need for static testing and inspection groups and for the development of design and constructional methods to be adopted as standard practice by industrial and commercial operators in the field of constructional steelwork and as protection for clients ;

(h) the urgent necessity for large-scale investment in production facilities in order to provide a complete answer to the problem of the prefabrication of load-bearing members, cladding and finishes, and thereby lay the foundations for a wider use of lightweight structures embodying standardized structural components;

(i) the need to re-assess and clarify corrosion phenomena, including their economic aspects, in order to develop standard methods of protection which will meet requirements as regards technical aspects, cost, effect on materials in the environmental conditions involved, and durability.

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Marcel BOURGUIGNON

Recommendations from the Work of the European Convention in Framework for Large Buildings

(Original text: French)

Within the compass of the work of this Working Party, I should like to tell you of the anxiety of those builders who use steel framework and of the organizations who represent them in the face of certain aspects of the very difficult problem which confronts the user rather than the planner in steel construction work.

At present one is worried by the encroachment into the building market of reinforced and prestressed concrete, thanks mainly to prefabrication methods and to improvements in operational and organizational techniques.

Moreover, following the experience and example of the Americans, one would have expected that the constructors of the large buildings which sprang up after the war in the cities of Europe would have chosen steel. Unfortunately this did not come about and many of them built by traditional methods or with large or small scale prefabrication in reinforced concrete, despite the irrefutable structural advantages of steel.

One considers that one explanation of this disenchantment is the lack of information; the limited experience and education of architects in steel construction work, and the lack of consulting engineers specializing in this branch of building. It is clear that there is much to be done in this field, and this will be the subject of the third part of my discourse.

The economic aspect of the problem I have just outlined has been studied by many individuals and organizations in order to find a good basis for a joint conclusion, willingly advanced by us, namely to know whether the use of steel framework is indicated mainly for economic reasons for the construction of multi-storey buildings of a certain height.

The results of these studies have been compared in the report which I had the honour to present to the members of Committee XI (Large Buildings) of the European Convention of the Associations of Steel Construction Work.

Without going into the report in great detail, I believe that the conclusions will interest you. I quote:

"We can assert in a general way, on the basis of present studies, conclusions of studies already carried out and experience already acquired, that the difference between steel and reinforced concrete as solutions is low and frequently negligible vis-a-vis the cost of the finished building. A difference of a few percent on the cost of one large building which normally represents only 10-20% of the total cost is of virtually no importance.

The American study put special emphasis on speed of building, which is one of the main advantages of steel construction work. This advantage is translated into dollars as extra profit from quicker occupation. It also seems that for all the types of building planned this supplementary income based on an annual profit of $ 4 per square foot amounts to 5.5% of the total cost of the building, so that steel framework leads to an economy of between 1 and 20% of the total cost.

While this aspect of the problem is familiar and has been indicated in all the studies devoted to it, the advantage of a faster completion rate is rarely translated into figures. It would be interesting to do this basing oneself on conditions in the European market.

It is clear that for this advantage, which remains merely potential until there is a general swing to steel construction work, to be consolidated and become a real gain, it is essential that it is not cancelled out by delays in steel supplies. The steel manufacturers must realize that all efforts towards an increase in the amount of steel used in building will be in vain if we are not guaranteed regular supply, and if in exceptional cases we cannot benefit from preferential treatment.

The increase in income due to the smaller amount of space occupied by the units of steel framework should be calculated. We note that the Italian study has shown that at the lower levels of an 8-10 floor reinforced concrete building, the ratio between the total surface measured at the external wall surface and that occupied by vertical supporting structures is of the order of 3.5% as against 0.6% for a building with a steel framework.

From the architectural point of view it is generally recognized that steel enables buildings to be constructed, that are structurally and aesthetically correct as the Mies van der Rohe school has shown. More particularly, steel can be made into girders whose depth does not exceed that of the floors, and this will provide further developments in prefabrication and a better use of the curtain-walling technique.

Paraphrasing these conclusions, we can say that on the economic plane a rough comparison of the cost of a steel and a reinforced concrete framework is not decisive and is not significant in regard to the total cost of the completed building. Moreover, the speed of completion shown recently in a number of cases, especially in my own country where I can quote the Hotel Westbury, the Tour Madon and the Ieteren Building in Brussels, the Albert Centre in Charleroi, and the Delta Building in Mons, is an advantage which can lead to a ratio clearly in favour of building with steel. We shall doubtless regret that the cost comparison is not generally known at this time.

Another aspect of the problem is the present tendency to combine steel and reinforced concrete in the construction of large buildings. We know that their great height makes them vulnerable to wind action so that it is necessary to plan the structure to resist this efficiently. In order to retain the isostatic nature of steel framework and thus reduce cost, two solutions are generally adopted: the horizontal action of the wind is absorbed by vertical steel bracing girders, or by a concrete central core housing lift shafts and services.

This second method of building seems to be regarded more sympathetically particularly since the introduction of sliding falsework which has speeded up the completion rate. The quickly erected core can be surrounded by a light isostatic steel framework. The most spectacular examples of this method are now in their final stages in Brussels, the Madon Tower, and in Charleroi, the Albert Centre.

Despite the efficiency of this method, it could be asked whether it is not a restraining factor on the architect. It implies in fact a radiating framework with rooms outside the core without any other alternative. A regrettable restraint on plastic form for the architect.

On the other hand the vertical bracing designs permit the well-known Phoenix Rheinrohr type of construction in Dusseldorf. Nevertheless the correct installation of these vertical frames necessitates close collaboration between the rachitect and engineer from the pre-planning stage. This point has been laboured enough, so I shall not reiterate it here.

We believe however that from the calculation point of view something remains to be done, mainly in the adoption of vertical steel bracing. Indeed up to now the maximum flexure limit of these girders, which are sunk in the ground and are free at the top of the building, has only been empirical. The problem is a complex one. Indeed the optimum dimensions for these girders under the effect of wind and the depreciation permitted by the regulations lead us to accept amounts of flexure which, if they occurred, would be incompatible with the good behaviour of the building in its upper storeys. These maximum distortions do not in fact occur because of the rigidity of the whole building.

As far as one knows this latter factor does not enter calculations because it is too difficult to estimate.

This problem ought to be the subject of a study conducted to a norm.

In conclusion, I should like for a moment to return to the problem of information and education in steel building on which I touched at the beginning of this talk.

We know that for many architects this lack of education and experience in building with steel is such a handicap that often despite their wish to have recourse to it, they very often abandon this method of building in favour of reinforced concrete.

We should note that they are not generally helped in this direction by consulting-engineers who also find themselves perforce better qualified in reinforced concrete than steel. I believe that this present situation in my country is general. I know of a recent example when an industrialist who wished to build a works and offices in steel called in an architect equally in favour of this type of building. The consulting engineer, who was better qualified in reinforced concrete, succeeded in convincing both client and architect that it would be easier to adopt the method he recommended.

The steel information centres, like the Belgian-Luxembourg Centre, are worried by this problem, and have for many years placed research and information services at the disposal of architects and consulting engineers. We have happily had good results in our country as a consequence of this.

We recently opened a Central Research Bureau with the co-operation of the topmost scientists in the country which will both study and carry out research on all problems of steel construction work.

This course ought, however, to be repeated in the field of instruction.

The higher architectural institutes in Belgium were contacted from our side. With the co-operation of the teaching body we attempt to interest the students in steel building by lectures, conferences, guided visits, and also by the establishment of competitions.

We believe in the matter of consulting engineers that in some faculties a more specialized education in steel building would be greatly appreciated.

Too few engineers are interested in or specialize in steel building, and if they are, they practice their teaching in workshops.

We believe then that as a means of information and propaganda, an international competition, perhaps biennial, to reward the best architectural work based on the use of steel, would be welcomed.

I address this suggestion mainly to the organizers of this Congress, believing that in this field the High Authority is the organization best fitted to patronize such an undertaking.

Guy BLANCHARD

Constructional Steelwork

(Original text: French)

The structural fields of interest to our profession are many and varied. Among those which spring to mind are:

— bridgeworks,
— industrial buildings,
— administrative buildings,
— structures for fitting out schools, hospitals, recreation centres,
— dwellings.

We will deal quickly with the first four sectors mentioned, in which our position, although breached with some success by competitive materials, has never ceased to be comparatively strong.

Steel bridges, at least those of relatively large span, can easily bear comparison with reinforced or prestressed concrete bridges.

Industrial sheds represent a field which for the moment is almost entirely reserved for us.

In office buildings, where fittings are gaining importance at the expense of the structure, steelwork is increasing in its appeal because it facilitates the use of decking and cladding in which service ducts may circulate. Besides, it offers unchallenged possibilities for alterations. The space saved by its use is also advantageous.

As for school buildings, the success achieved by prefabricated steel structures is so well known that it is unnecessary to dwell upon them further.

We therefore come to the last of the fields mentioned at the commencement of this paper, that is dwellings, which I propose to consider at some length.

Where are we in France at this moment?

Of the 320,000 dwellings or so being built annually in France, how many have steel frames? We know the answer only too well. However, France is short of houses and it is desirable that the rate should reach half-a-million per annum. The general interest of the country therefore appears, in the emergency, to coincide precisely with that of the steel and structural steelwork industries.

How, in these circumstances, can we explain the situation, at first sight paradoxical, which we know?

We can think of reasons outside our profession. For example, it is customary, in this field, to speak of the delayed reaction of the French public which is temperamentally opposed to all innovations. Property legislation has also been equally blamed.

The Steel industry has also been taken to task. In 1945, for reasons beyond its control, it could not provide the fabricators with the tonnage of steel required to promote the use of structural steelwork in dwellings. The Korean War did not improve the position.

The fabricators themselves have not been spared criticism. They have been accused of ignoring housing, except as a subsidiary activity to fill in the gaps in their order books.

But this is not the whole story. It is quite certain, however, that the dearth of steel after the last war gave no chance to the fabricators to assert themselves, even at a time when a mere 40.000 dwellings were being erected per annum.

Nevertheless, this does not seem to explain the present inactivity.

The public is not reaily as hostile ta new ideas as we are led to believe and this Congress is a proof of the interest which the steel and structural steelwork industries take in housing. It should also be added that the public authorities now give a not inconsiderable degree of support to the promotion of structural steelwork.

Under these circumstances, the least one can say is that it would appear that steel is assured of a reasonable place in the construction of dwellings, despite the fact that this is not the case at present.

Why? I believe that having reached this stage in our deliberations, we must cast off all preconceived notions and ask ourselves this question:

Are the traditional structural systems which we offer, technically and economically competitive in today's markets?

I believe one can only reply in the negative.

There is, in fact, no instance of a worthwhile product, the need for which is felt by a large number of customers, which does not find a buyer.

By contrast, we know only too well from everday experience that traditional structural steelwork, consisting of colunms and beams of H-, or I-section, in which lateral stability is given by wind-bracing or fixity of joints, does not provide the modern answer to the housing problem. Although some local, isolated success may be achieved, ignorance of this fact, when entering the French housing market can only result in set-backs.

This applies not only to detached houses but also to blocks of 3,10,20 or even more storeys. Although it is often claimed that a structural steel frame is competitive above ten storeys in height, I am not convinced that this is so.

How have we arrived at such an apparently unfavourable position?

I believe that we can find the answer to this question in a comparison of these two figures:-

— 40,000 dwellings built annually in the years immediately after the war.

— 320,000 dwellings built annually now, in which structural steelwork makes a modest contribution.

This eight-fold increase in the number of dwellings built has been made possible by the extraordinary progress achieved in the building industry. This, together with the fact that the French school of reinforced concrete is without argument one of the best in the world, explains why we cannot fight with weapons more than 20 years old against extremely competitive systems which are even exploited in the U.S.S.R.

One could argue that we are still very far from the half-million dwellings desired annually. The following reasoning has an evident appeal:

> "Competitive or not, as there are still 180,000 dwellings which cannot be built by our competitors, you must seek our help in every way."

Such an argument has no value, in my opinion, as I am convinced that an industry which has increased its output from 40,000 to 320,000 dwellings could easily achieve 500,000. The serious lack of labour we now suffer is undoubtedly only a passing phase.

Under these conditions, should we give up the struggle for a share in the construction of dwellings?

Such an attitude of resignation would be as misplaced as a desire to fight with obsolete weapons. It must be realized that if we allow our competitors to appropriate the construction of dwellings without a struggle, the day may well come when we will no longer be able to defend ourselves in those sectors we now consider to be reserved for us. The economic and academic potential which we will have allowed to grow at our expense could well weigh heavily against us in competition.

It is therefore essential that structural steelwork should emerge from its isolation in this field.

For that reason, an extensive investigation must be undertaken by assembling, if possible, all our possible resources.

In fact, the problems to be solved are too complex for any one of us to hope to solve them alone.

Without pretending to lay down a programme of investigation, I believe that we could profitably study the following subjects:

Floor beams

The light rolled steel joist usually associated with concrete slabs is hardly ever economical by virtue of the fact its use necessitates the provision of a ceiling, the price of which is too high. The quest for an economical ceiling is therefore essential if we wish to preserve the rolled steel joist.

A solid reinforced concrete slab incorporating a high yield stress steel framework made from sheet sections could provide a composite form of construction which was economically and technically viable if a system of shuttering were also developed at the same time. In the latter field a great deal still needs to be done.

Permanent sheeting incorporating reinforcement for the slab also appears to be a worthwhile subject for study. To avoid the need for a ceiling, the appearance of the soffit must be satisfactory.

Beams

Except in particular cases, the conventional rolled beams are also uneconomical. Joints are difficult to make while the use of deep sections to overcome the deflection problem results in considerable drawbacks, there being complications in fitting the ceiling or a loss of headroom.

The solution may lie in the manufacture of unsymmetrical sections produced by welding and incorporated in the concrete slab.

Of course, the suggested solid slab mentioned above resolves this problem.

Stanchions

As a general rule, these ought in principle to be loaded only by vertical forces. A frame with rigid joints, except in extraordinary cases, cannot be compared with competitive alternatives. It is therefore necessary for us to study a combination of steelwork and concrete such as that which has been constructed in the Place Madou, Brussels.

The use of H-sections in mild steel no longer leads to a very competitive design. One should probably consider the use of stanchions in high yield stress steel fabricated by welding up either a box or an H-section.

Cold formed sections also appear to be an interesting possibility for the production of tubular stanchions, hollow or filled with concrete, for use in low buildings.

This brief summary of the problems we must solve shows, I hope, the importance of the route we must follow to integrate constructional steelwork into the sphere of dwellings.

It is indeed an integration because, when we bear in mind the respective strenght of each, it would be absurd to assume that we could replace the traditional builders. I believe that we can make a loyal contribution to the work in this field.

I am convinced that this collaboration would be profitable to each one of us and to the general advantage of all.

Fernand ROCHEZ

Towards a Possible Solution and an Adaptation of Structural Steel Frames in Tower Blocks in Europe

(Original text: French)

The design of a tall building must conform in a definite manner with a range of criteria which I shall try to define.

For traditional buildings, of less than 8 storeys or 80 ft. in height, the incidence of the design and choice of structure is roughly one-sixth of the cost of the scheme, while the structures and foundations for tower blocks (250 to 330 ft. high and more) represent round about one-third of the total cost. In addition, the aesthetic influences in direct relation to the structures, the space taken up by the latter in the plan, the requirements of special techniques (heating, ventilation, electricity), are determining factors in direct relation to the skeletons and represent the most important element in the success or otherwise of the structure as a whole. It would be rash to claim that a single solution exists, even in steel, for all the possibilities of prefabrication, of structures or any other works.

The more *outré* ideas vaunted by certain project designers remain mere flights of fancy and have never caught on in practice. Nevertheless, what efforts have been made to evolve this final universal panacea, the construction that will be fully economic and that will meet all the requirements involved in the completion of a building ?

It is physically impossible to invent anything in this respect, but we must adapt and sometimes try to innovate — this is without doubt the best way to make intelligent progress really in line with the needs of the times.

It is only by analysing all the considerations involved and by drawing conclusions that a suitable choice of structure can be properly and objectively made. These considerations are :

(1) Ensuring the stability of the building.

(2) The location—often an urban site—the high cost of land and limited superficial area available.

(3) Nature of the soil—generally land which has already been disturbed, requiring special or deep foundations.

(4) Economics of the scheme and quality of the structure.

(5) Ensuring a pleasing appearance.

(6) Adherence to the plan and proper functioning thereof (dimensions).

(7) Incidence of special techniques.

(8) Availability of labour.

(9) Time required or taken for construction (capital investment).

(10) Durability and behaviour under atmospheric conditions and in the event of fire and from the point of view of acoustics, convenience, etc.

The problem is not an easy one and only a study of practical cases will enable us to appreciate the difficulties. Let us consider two examples :

The Centre Albert, Charleroi
(tower block, 280 ft. high, for use as offices)
main structure completed (1 to 6).
The Parking Hotel, Charleroi
(multi-purpose building, shops, parking facilities, offices, flats, hotel, etc.)
still in the project stage, (7).

These two schemes have been successfully launched, with excellent co-operation from architects, engineers and contractors alike, thanks to the enterprise and energy of

Mr. Jean Baudoux, the promotor of both schemes, to whom I should like to take this opportunity to pay tribute.

The Centre Albert

Brief description :

- — height : 280 ft. (25 storeys + 2 storeys for technical installations) ;
- — total superficial area : 225,000 sq. ft. in reinforced concrete ;
- — Foundations on Benotto piles of 400 tons in mining area;
- — general lining of basements ;
- — structural steelwork : encased beams, stanchions in high yield stress steel ;
- — Kaiser prefabricated flooring ;
- — sliding shuttering for wind core and steel shuttering for cross walls. See appended quantities and tables.

Six possible schemes were compared.

The structural steel scheme comprising prefabricated ribbed flooring and central concrete core with sliding shuttering was chosen for the following reasons :

(1) Economy: despite a relative handicap of 10 per cent on the scheme in concrete alone.

(2) Speed of construction: less than one year for the whole main structure; 25 days for the central core; 3 storeys per week for the structural steelwork.

(3) Small but specialized labour force.

(4) Lightness and pleasing appearance: grid 20 ft. by 23 ft.

(5) Little storage of material required on site.

(6) Lightness and flexibility: movement in mining area.

Review of costs : for 39,000,000 Bf. for framework 17,500,000 in steel (sections, reinforcement bars) instead of 9,000,000 in reinforcement for concrete frames.

This is a real and effective advance in steel utilization.

Parking Hotel

Description :

- — Height : 345 ft.
- — Volume : 2 million cubic ft.
- — Total superficial area: 322,500 sq. ft. of which 280,000 sq. ft. is useful area.
- — Foundations direct on sandstone at 2nd basement level; complete lining of basements ;
- — structural steelwork : encased beams, stanchions in high yield stress steel ;
- — prefabricated ribbed decking ;
- — two central cores with sliding shuttering, joists being used as tension reinforcement.

The number of cores was determined by stability considerations and by vertical circulation (lifts, air-conditioning ducts, etc.)

It proved impossible to design the frame in reinforced concrete because the excessive dimensions of the vertical columns would have prevented the proper functioning of the lower storeys. Although the possibility was interesting, in the event steel flooring was not used. In fact, this is mostly used in dwelling and so has to meet even more requirements.

The advantages are the same as in the Centre Albert. Moreover, we expect that the time required for the actual construction itself will be even less. Those interested will be able to examine the comparative tables, the estimated cost of the different schemes and the appendices relating to execution and construction principles.

This design has shown that the introduction of sliding shuttering is a real advance in steel utilization. Thus,

(1) the complete framework usually requires cumbersome and costly wind bracing.

(2) Formerly, concrete cores, shuttered in the traditional manner, considerably slowed down construction which could not forge ahead for lack of support, thus reducing the possibility of using steel skeletons.

(3) The combined use of a steel frame with a concrete core in sliding shuttering offers the following advantages :

- (a) same rate of construction (10 to 13 ft. per day in the Centre Albert) for both the steelwork and the concrete, which in this case comes first;
- (b) maximum tolerance of approx. 3 mm. for the core, resulting in simple adjustment in the steelwork;
- (c) natural simplification of the structures — wind completely absorbed by the core — the stanchions are subjected to axial compression and the beams link the stanchions and walls — the joints are simplified, there being no rigid connections — rationalised foundations;
- (d) gain in useful area;
- (e) large proportion of prefabricated steel members, reinforcement, stanchions and beams;
- (f) minimum amount of labour required;
- (g) rapid setting-up of the lifting equipment (cranes, hoists, etc.) which rise with the central core;
- (h) direct introduction of the vertical and horizontal ducting in the core and framework as and when required as construction proceeds;
- (i) Economy-speed : In a scheme being studied for a similar tower to be used as offices, we contemplate the possibility of using steel flooring with flexible linings. It seems that this construction is competitive as it has been used notably in the Delta building in Mons (architect Mr. Lavendhomme), and in the Ieteren buildings in Brussels (architect Mr. Stapels) in all-steel frameworks the method has given complete satisfaction.

The undeniable success of the use of steel in building must be obvious, not only by the originality of the solutions so rapidly produced for numerous problems, but also by their suitability for use from both the technical, the economic, aesthetic and social standpoints.

CENTRE ALBERT, CHARLEROI — OFFICE BLOCK

(1) Height: 85 m.

(2) Total surface area: 21,000 sq. m.

(3) Steel frame: 890 tons of steel section

(4) 3,938 cu. metres of reinforced concrete comprising:

(a)		1,523 cu. m. for core (sliding shuttering)
(b)		450 cu. m. for external cladding (metal shuttering)
(c)		842 cu. m. of beam encasement (fire protection)
(d)		900 cu. m. of foundations
(e)	±	300 cu. m. slabs and misc.

(5) Floor of Kaiser type precast ribs: 18,000 sq. m.

(6) Cost of structures and foundations: ± Frs B. 39,000,000
breaking down as follows:

(a)	±	10,000,000 (wind bracing)
(b)	±	10,000,000 (floors)
(c)	±	11,500,000 (steel frame)
(d)	±	7,000,000 (foundations and misc.)

giving Frs. B. 1,850 per sq. metre, made up of:

(I)	Frs B. 500 for floors
(II)	500 for the frame
(III)	500 for the central core
(IV)	350 for the foundations

(7) Steel used (materials):

± 11,500,000 (hot-rolled steel sections)
± 3,500,000 (concrete reinforcement bar)
± 2,500,000 (precast ribs)

making 17,500,000 (nearly half the total cost)

(8) A reinforced concrete structure of the same size would require ± Frs. B. 9,500,000 of reinforcement bar alone and this for less floor space.

CAR PARK HOTEL — MULTI-PURPOSE BLOCK

(1) Height: 105 m.

(2) Total surface area: ± 30,000 sq. metres, giving 26,000 sq. metres useful floor space.

(3) Steel frame: ± 1,300 tons of section

(4) 8,690 cu. metres of concrete comprising:

(a)	2,490 cu. m. for cores (sliding shuttering)
(b)	2,200 cu. m. for ramps and slabs
(c)	1,500 cu. m. of beam encasement
(d)	2,500 cu. m. of foundations (2 basements)

(5) Precast rib floor: 22,000 sq.m.

(6) Cost of structures and foundations: ± Frs. B. 60,000,000
breaking down as follows:

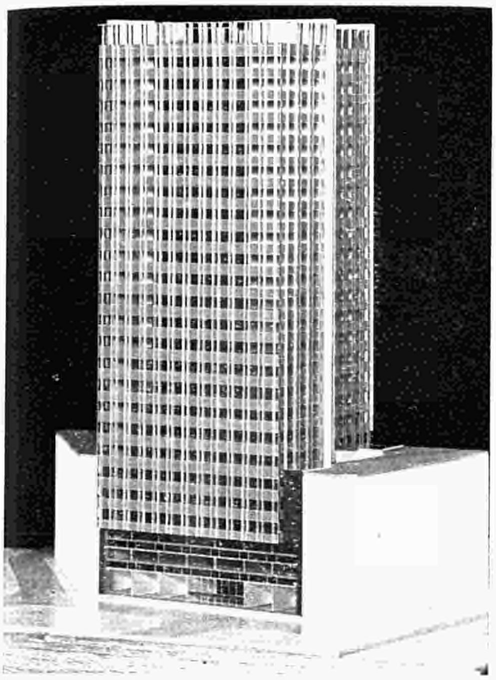
(a)	±	14,000,000 (wind bracing)
(b)	±	19,000,000 (floors, slabs and ramps)
(c)		17,000,000 (steel frame)
(d)		10,000,000 (foundations)

(7) Steel used (materials):

17,000,000 (steel frame)
6,000,000 (reinforcement bar)
3,500,000 (precast ribs)

26,500,000 (nearly half the cost of structures and foundations)

(8) The possibility of a reinforced concrete construction could not be considered because of the excessive bulk of the basic elements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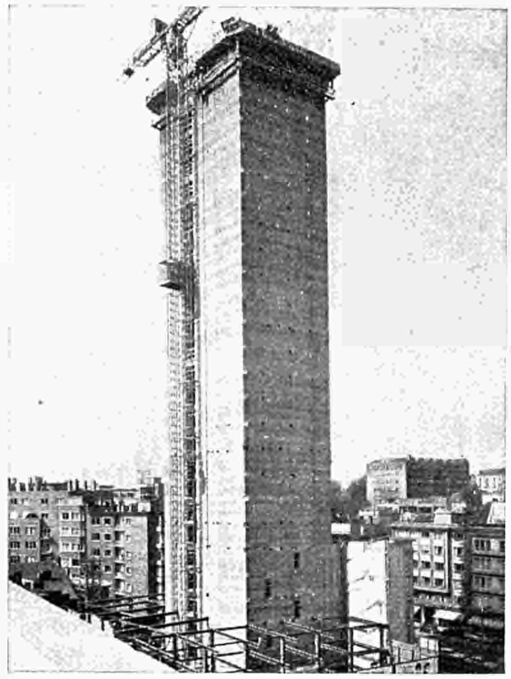
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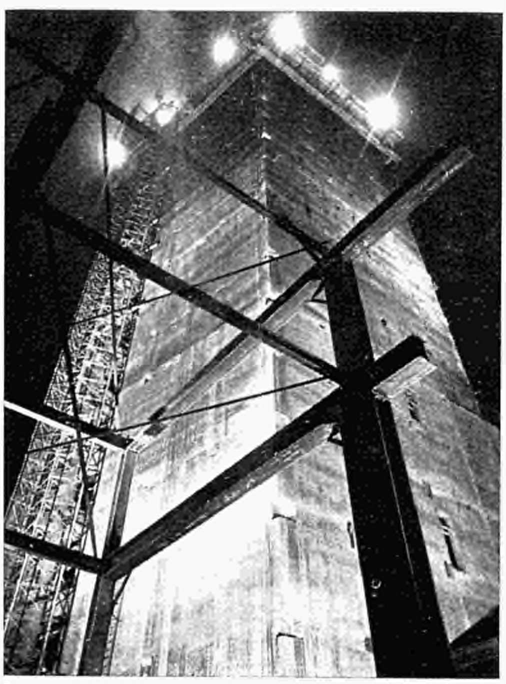
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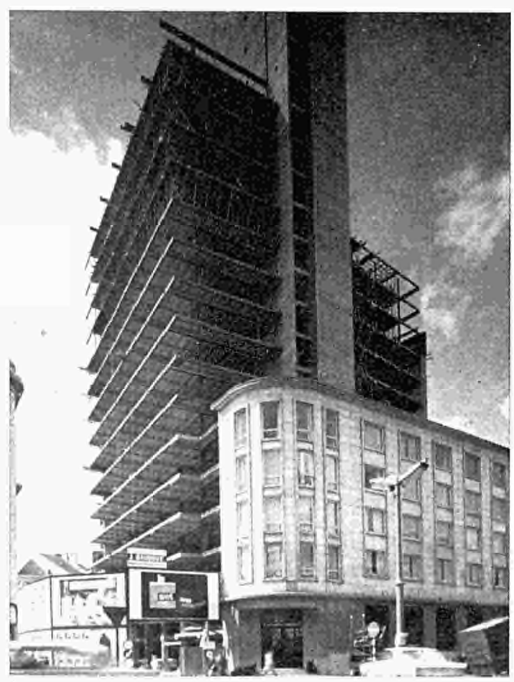
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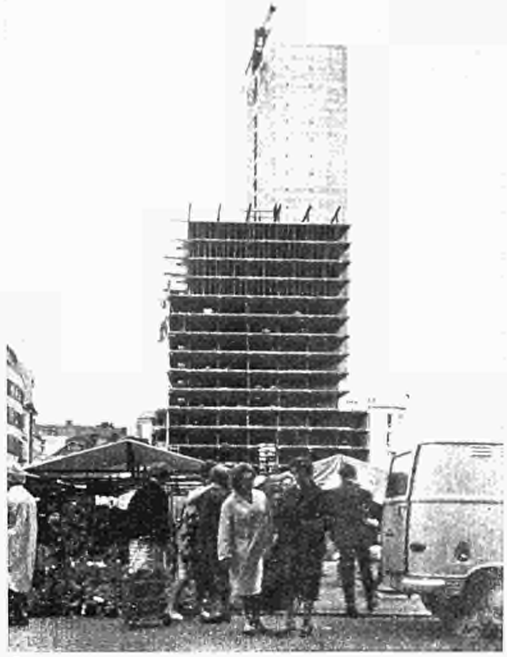

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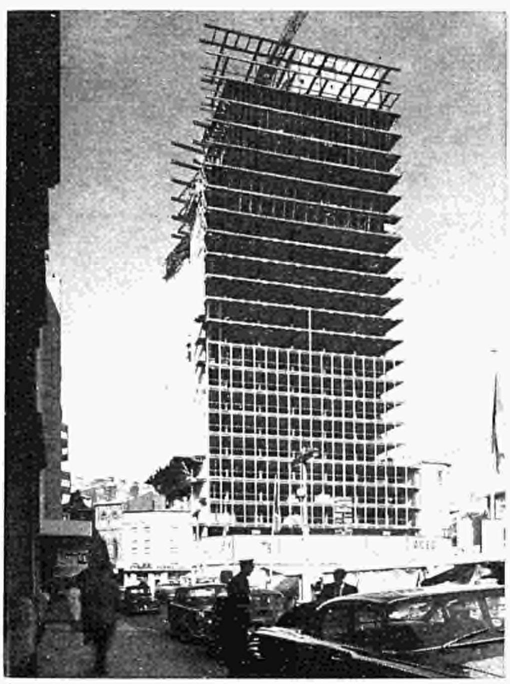

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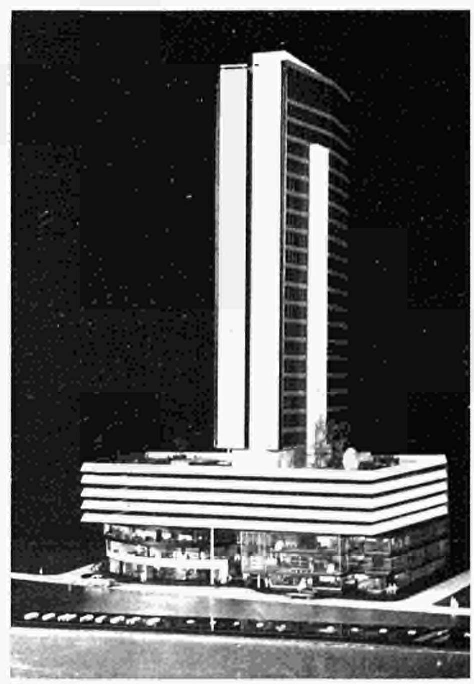

7

Werner BONGARD

Advantages and Problems of Steel Construction

(Original text: German)

The papers presented at today's session have provided us with a comprehensive balance-sheet showing the current position of steel construction. The elements in the balance-sheet apply, generally speaking, to my own country as to the rest, and I am therefore, (with the Chairman's agreement,) not proposing to dwell in detail on steel construction in Germany.

Looking first at the credit side, we find that building in steel offers convincing advantages, based on both structural and functional arguments. By the precision of measurement which it permits, steel is well suited for all types of building incorporating prefabricated components. Steel construction is indeed the classical mode of prefabricated building. It has long been the practice to produce standardized structures on the principle of interchangeability. The structural steel industry has in addition observed the trend of the times and is utilizing the still largely untapped potentialities of mass production in automated phased production lines. The market situation demands that the client should be supplied with buildings ready for immediate use; this is taken for granted in the case of standard structures, and is becoming more and more common also with the prefabrication of components. Our main task — as has been pointed out by several speakers — is to present the case for steel to the architects and to the rising generation; precisely how this should be done ought perhaps to be discussed.

Over against these credit items on the balance-sheet, are various questions and problems to which our present and future efforts must be directed. I should like to enumerate a few points. First, ceilings. The ideal would be a steel ceiling, which provides both the necessary safety platform for further erection operations, and at the same time fulfils all functional requirements. However, the cost is unfortunately still a problem here, so that it is usual to employ composite ceilings of steel and precast concrete slabs.

Further research will be necessary in this connection. Also, we shall have to see how the price trend develops: some day we may be able to install cellular steel ceilings at attractively low cost.

Another aspect that is very much to the fore is the question of fireproofing. The problem itself is the same in every country, but unfortunately the point of departure differs widely, as the individual countries have their own separate regulations, and some indeed do not even have homogeneous regulations within their own borders. It would be splendid if we could use the simple formulae which Dr. Kollbrunner has given us, but unfortunately things are not quite so simple. Basically, the position is that where the fire load is slight, nothing needs to be done. It is however necessary to prove to the satisfaction of various bodies that the fire load really is as slight as that, and will remain so throughout the lifetime of the structure. So it is a question of verification.

One last point. Single-storey shed-type buildings do not as a rule need fireproofing, at any rate according to our regulations in Germany. But where, for specific reasons, fireproofing is necessary, it is desirable to use fireproof paint specially made for use with steel and also not liable to spoil the architectural design. We are already at the stage where we can offer a paint described as "fire-resistant." I consider this a tremendous help. We are now waiting to see what further requirements can be met with the aid of this product.

Gavin Burton STEWART

Tubes in Structural Framework

(Original text: English)

A discussion on structural steel framework would be very incomplete if the structural tube were not mentioned.

You are all aware of the qualities of a tube as a structural member. Its lightness, high torsional resistance, resistance to buckling, aerodynamic qualities, ease of protection, natural elegance, etc., are well known, and I do not propose to go once again over this familiar ground. More pertinent is to try to explain why, with all these admitted advantages, tubes do not as yet constitute the backbone of steel structures.

The answer is twofold. The first is a question of cost. Tube is, generally speaking, more expensive than solid rolled sections, though it is perhaps not widely appreciated that this is usually more than compensated by the lightness of the tubular structure and its other advantages. The second answer is that the tube is suffering from the legacy of a difficulty — that of making satisfactory joints. As long as bolts and rivets were the only means of making joints, the tube was handicapped by the inherent difficulty of its curved surfaces. Thus, though the virtues of the tube were well known and indeed embodied in some splendid engineering achievements, the tube, present everywhere to convey fluid, was seldom used as a structural element. Modern welding technique has fully and effectively resolved this difficulty and the tube has begun to occupy its rightful place in steel structures.

To further this process, the leading tube makers and tubular constructors of the world have formed an international association to pool their knowledge and experience, supplement these by appropriate co-ordinated research and join forces in disseminating reliable technical and economic information about the structural uses of tube. This organisation, of which I have the honour to be President, is the Comité International pour l'Etude et le Développement de la Construction Tubulaire, known as C.I.D.E.C.T.

Its headquarters are situated in Paris at 30, boulevard Malesherbes, and its technical centre at Park House, Park Street, London.

Much of the Committee's technical investigation centres on the technology and theoretical analyses of the joint. The indications are that the plain tube-to-tube joint, without any webbing or reinforcement, makes an ideal bond which is clean and simple as well as sound. It possesses "fixity" — what the French call "encastrement" — a characteristic which should allow for a substantial reduction of the effective length of compression members and thus add further to the lightness of tubular latticed structures. This can be illustrated by the case of lattice girder without lateral bracing. When such a girder is being loaded, it will tend to become laterally unstable, a phenomenon covered by the French word "déversement", and the better the fixity of the joints, not merely in the plane of the girder but also in the transverse plane, the better the girder. Here the tubular joint, with its excellent fixity in the transversal planes as well as the girder plane, scores heavily.

This is a complex technical matter to which I can hardly do justice in a few words, and indeed there is still a lot to learn in this field, but you may like to know that this problem of the fixity of the tubular joints and the associated effective lengths of tubular compression members is being investigated on a large scale by C.I.D.E.C.T. Research is being carried out on this at the present time at Liège, Karlsruhe, Turin, St. Rémy-les-Chevreuse near Paris, and Cambridge and all these test programmes are financed, directed and co-ordinated by C.I.D.E.C.T. I do not doubt that from this massive research there will emerge results which will vindicate that the tubular joint, once a problem, is now one of the chief assets of tubular structures.

I feel sure that, partly as a result of this international research, the steel tube will be called on to play a leading part in the structures of the coming age. Constructional steel will be pitted against the flowing elegance and adaptability of dlastic and concrete, and the simple, light, slender yet strong and sound tubular frame may well become the great champion of steel.

Jean BENOIST

The Steel Tube, the New Structural Section in the Service of Steel Development

(Original text: French)

The steel tube made its appearance as a structural section before the first World War. It was used in wartime to conserve steel, and tubular structures are now being erected all over the world.

It was once thought that the potential saving in weight would make the tube a competitor of other steel products. This is no longer held. In fact, the tube is just a new structural section which has come to take its place beside others, not replacing them but completing the range of structural sections and through its advantages, to a new expansion in the use of steel.

Some eminent speakers told us yesterday that the economy in weight achieved by the use of high tensile steel should not be considered a bad thing in itself as it constitutes technical progress. The same can be said of the steel tube.

A saving in weight, together with better appearance and reduction in the total cost of construction, are the major attributes of this type of construction when the section is used judiciously.

As Professor Cohen has said, it is known that steel with its characteristic resistance to torsion, bending and tension, has replaced the traditional materials resistant only to compression. The range of mechanical properties is now supplemented by the properties of the tube, which is especially resistant to compression and torsion.

Traditional rolled sections alone cannot provide a complete solution vis-à-vis competitive materials. However, tubes used together with rolled sections provide the answer to otherwise insoluble problems. The use of tubes therefore results in an increase in the use of steel.

Let us consider a few examples:

A very tall reinforced concrete building can have columns so stout that a point can be reached where the use of concrete is no longer possible owing to the restriction of space in the corridors and at ground floor level. The lightness of tubular steel construction increases floor space and reduces the size of foundations, the cost of which can otherwise be considerable. Typical examples which may be cited are the apartment block in the Rue Croulebarbe, Paris, and the Mannesmann and Phoenix-Rheinrohr office blocks in Düsseldorf.

Tubes offer considerable advantages in the construction of large-span industrial buildings. There is a reduction in dead weight of 50 per cent or more, transport and erection are easy, prefabricated members can be used and work can actually be carried out on the site, if so desired.

The Cartonnerie Devoiselle at Melun in France can be mentioned as a typical example. At London Airport, the first B.E.A. hangar was built in reinforced concrete, but the second, with a span of 40 m. (133 ft.) was built with tubular steel frames.

It had been thought that tubes could be used only for large spans, but the roof trusses 20 m. (66 ft.) in span, as in the Usines Berliet in Bourg, are interesting because of their ease of construction and on account of their low price.

We can say that, in many cases, structures making use of tubes have replaced buildings in competitive materials. It has often fallen to my lot, however, to be approached by consulting engineers who wish to have a tubular design alternative to one in concrete. Unfortunately, the original design has often progressed so far that nothing can be done about it.

In addition to the advantages I have mentioned, tubular steel construction has a number of others, such as requiring less maintenance (a point which lowers the overall expenditure involved, and is causing tubes to be much favoured by the chemical industry) and reducing the stresses caused by wind.

One must also mention the architectural advantages offered by steel tubes. As centering, with connections available in many directions, tubes facilitate the construction of vaults, girders of triangular section and three-dimensional structures, all of which are traditional shapes for concrete.

In France, examples include the vault of the Granval dam, the roofs of the swimming bath at Boulogne and of a church in Chartres, while in Germany there are many structures constructed by Mannesmann, in particular the church at Rath. For forther particulars, reference should be made to the January 1962 issue of *Architecture d'Aujourd'hui*.

The use of square and rectangular hollow sections should also be mentioned. Considerable developments are at present taking place in this field in the U.S.A., England and other European countries as a result of special studies which have been made to lower building costs.

It may be added that tubes can be supplied in any type of steel desired, including notch ductile steels.

In conclusion, the steel tube should be considered as an addition to the range of sections already existing. Its judicious amployment, that is to say limited to those structures or parts of structures which have been described, should lead to an increase in the total volume of steel construction.

While on this subject, I might perhaps mention that the Italian firm Costruzione Metalliche Finsider which follows these principles, uses from 15 to 20% of steel tubes, according to the particular type of structure.

The possibilities offered by tubes in the architectural field, with the adoption of shapes previously unobtainable in steel, make them an important factor in steel development.

Description of photographs

1 — Building in Rue Croulebarbe, Paris.
23-storey block. The stanchions are of concrete-filled tube and the beams of rolled steel section. The X-form wind braces are tubular. This example shows the combined use of tube and section.

2 — Devoiselle Company's factory at Dammaries-les-Lys.
The first building covers 13,000 sq. metres. The triangular main girders, 111 metres long, are supported. at two points 66 metres apart. The girders are 24 metres apart. The frame weighs 40 kg. per sq. metre

3 — Berliet factory at Bourg-en-Bresse.
Area covered: 100,000 sq. metres. The trusses have a 20-metre span. These are spaced at 3 metre intervals so purlins can be dispensed with. Weight: 11 kg. per sq. metre.

4 — Boulogne sur Seine Swimming pool.
The roofing frame is a three-dimensional structure taking the form of a parallepiped with a 50 metre side and 2.30 metres high. The structure weighs 30 kg. per sq. metre.

5 — "M.59" — 4.
Steel tube pylon, 115 metres high, for the 380,000 volt power transmission line across the Dordogne at the "le Marquis Marmagne" station.

6 — Saint Bernadette's Church, Dijon.
The tubular columns are 13.50 metres high.

7 — Massey-Ferguson factory at Beauvais.
Built with supporting monitors. Weight of frame: 19 kg. per sq. metre. The 30 X 12-metre roof sections were assembled and fitted in one piece.

8 — Church of St. John the Baptist at Chartres-Rechèvres (Eure et Loire)
A three-dimensional structure with a single skin. 24 metre diameter dome.

9 — Saint Charles station, Marseilles.
The length of the building is 74 metres in three bays. The tubular frontal girder is of triangular cross-section.

10 — Etablissements Neu research centre.
24-sided polygonal structure with a central tower. The V-columns are 293 mm. diam. tubes.

11 — Formont depot at Pantin.
The dissymmetric parabolic portal frames have a 28 metre span. The purlins are prismatic. The bays measure 10 metres.

12 — Chartres swimming pool.
The frame is of the three-pin arch type with a 22 metre span. The arch trusses are of varying inertia.

13 — S.R.E.M. factory at La Flèche.
Square shop (35 X 35 metres) built on the Pyramitec system.

14 — Onatra garage.
This building is 85 metres long. The arch trusses have an 18 metre span and the bays measure 5 metres. The frame weighs 14 kg. per sq. metre.

15 — Vallourec research centre, Aulnoye.
A 1,500 sq. metre building. The 100 mm. sided square tubes of the façace hold the IAP 200 joists of the first two floors.


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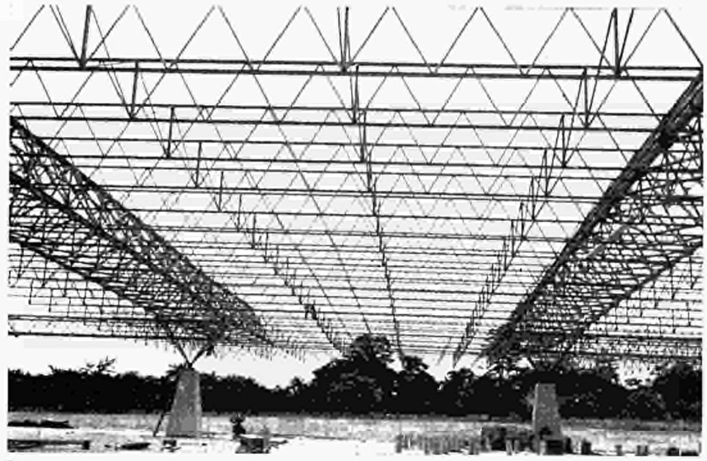
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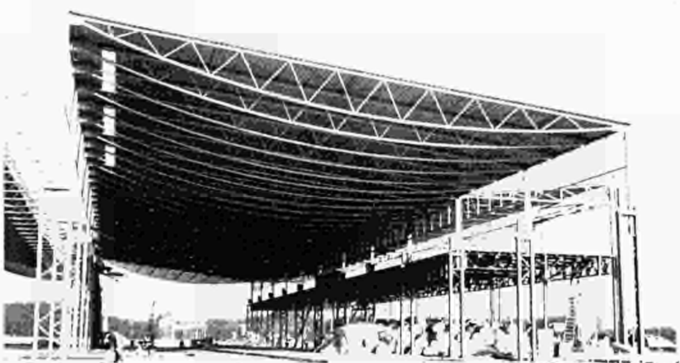
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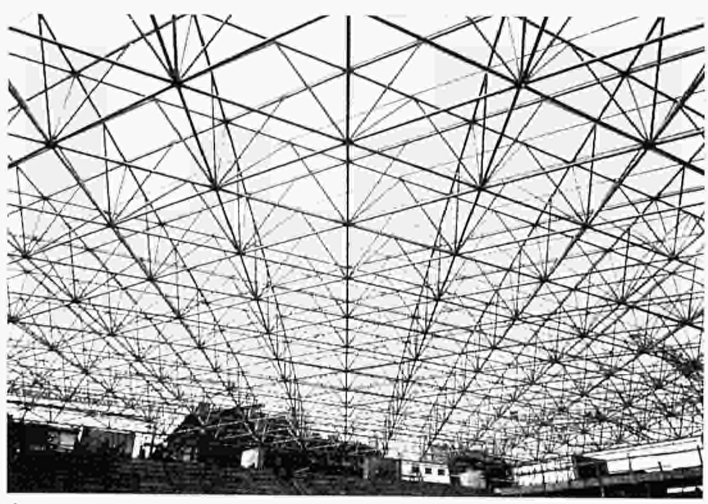
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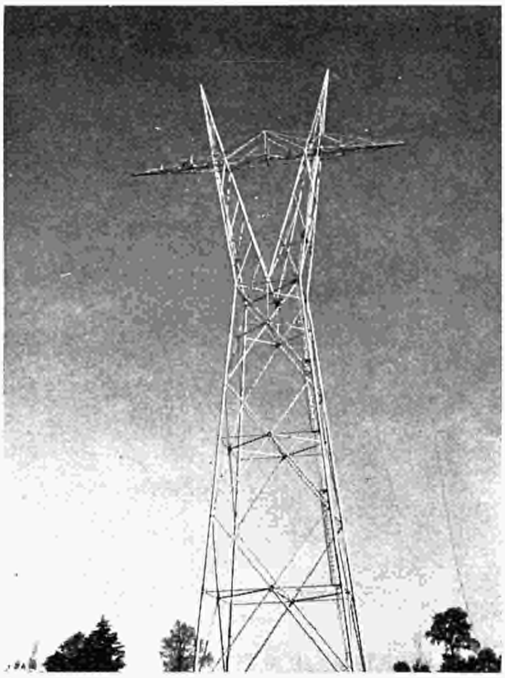
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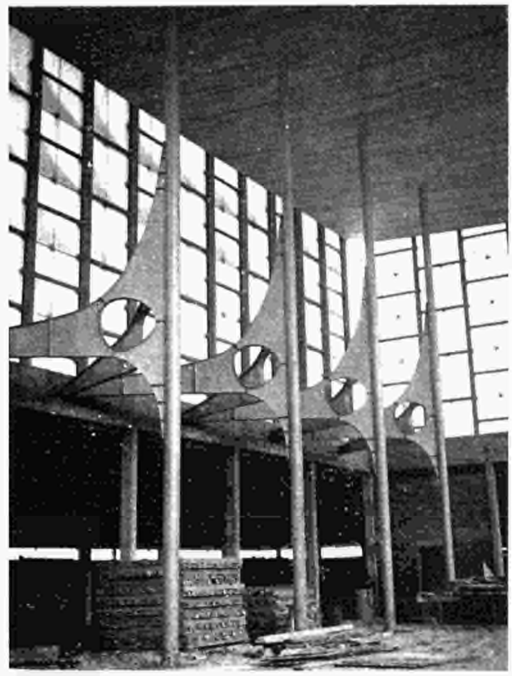
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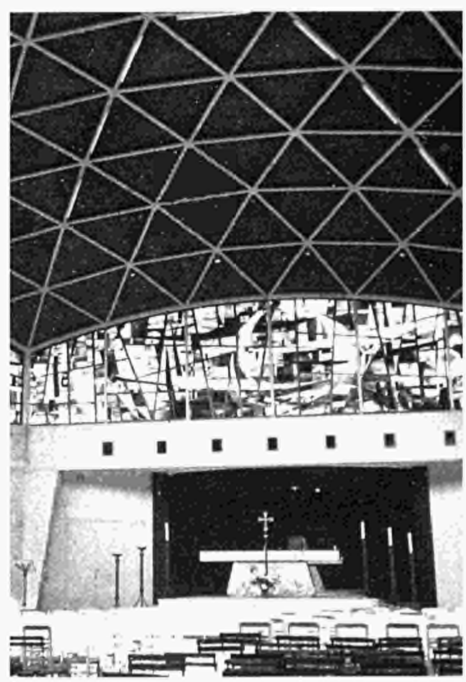
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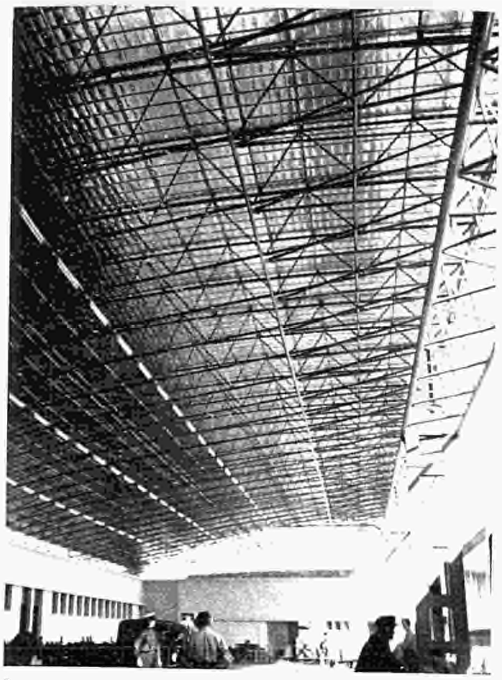
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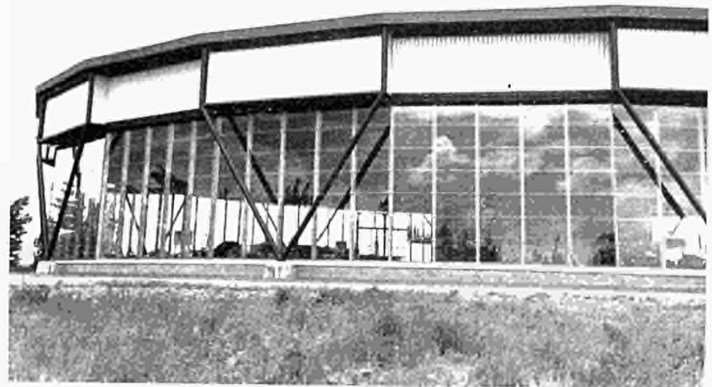
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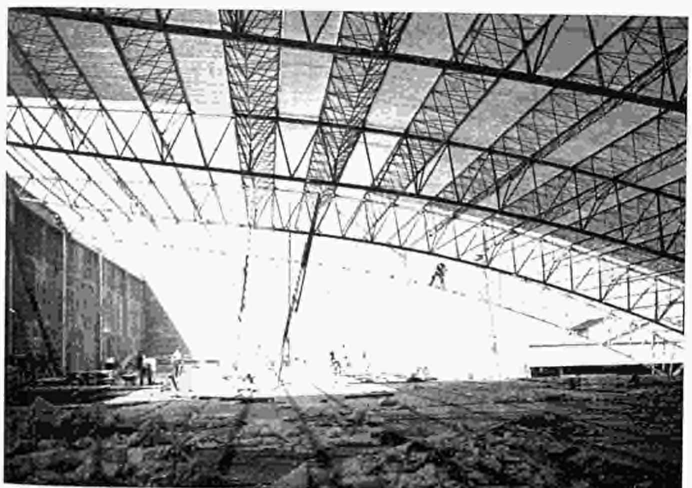
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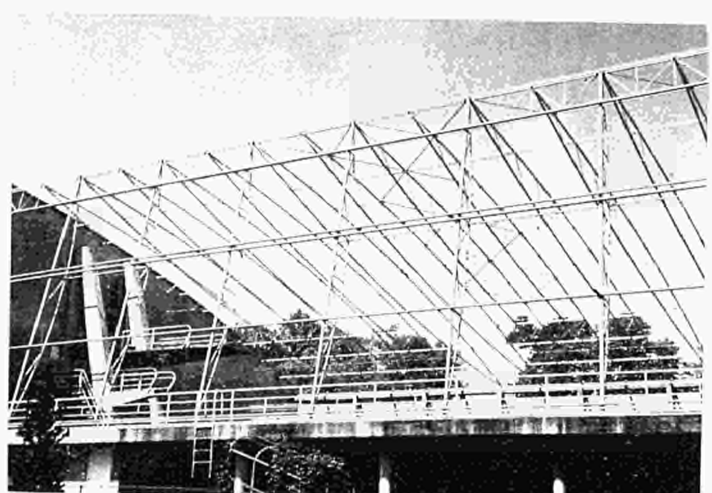
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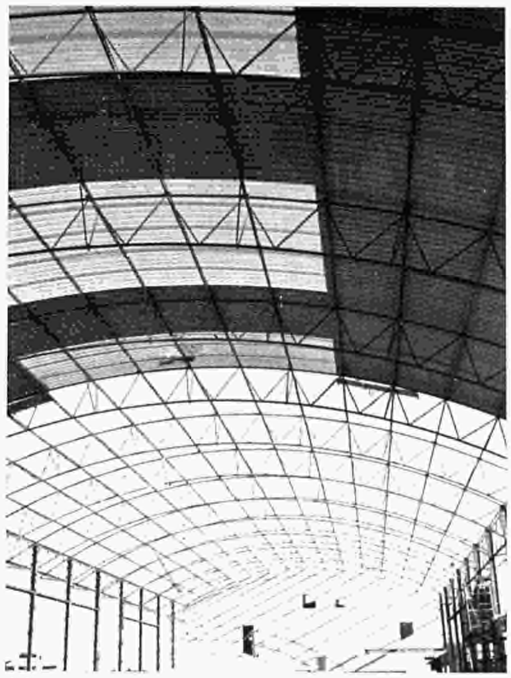
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René FORESTIER

A Revaluation of the Principles Adopted in the Construction of the Buildings for a Steelworks at Dunkirk

Further Observations on the Construction Phase

(Original text: French)

Most of the members of Working Party III know the problems which were involved in the construction of the USINOR steelworks at Dunkirk.

It will be recalled that the soil was of very poor quality, as the underlying compact clay was covered with a 32 m. (106 ft.) depth of sand which contained scattered pockets of silt of varying thickness and extent.

In addition, the corrosion already present on the side facing the sea, had extended due to the action of steam and smoke, from adjoining industries and from the steelworks itself.

When the problem of possible future extensions, was also considered, the choice of the best technical and financial solution could only be made after a complete examination of the whole question of foundations, framework and protection.

The alternatives which emerged were either a traditional column and truss construction, or portals fixed at their bases, or portals with hinged feet.

The steelwork could be latticed or of plate girder construction with stiffeners visible or in box construction with internal stiffening diaphragms.

It was also necessary to decide whether to paint the inside face of the plates and the internal stiffening of sealed hollow box portal frames.

In the case of fixed bases, no rotation whatsoever being allowed, it was necessary to consider either heavy concrete piles (susceptible, however, to horizontal movements due to the creep of the subsoil under working loads) or extremely wide slabs placed on compacted sand.

As for hinged feet, vertical settlement and horizontal displacement could be easily accommodated by a structure articulated for these conditions. In this case, concrete piles could be advantageously replaced by sand consolidated by either vibroflotation or vibrosinking.

Our examination of the whole question led us to choose portals with hinged feet, the design being rather special and taking account, in addition, of the problems of future extensions.

In cross-section, the buildings comprised main portals of even bays in box construction (generally reserved for heavy overhead travelling cranes) and sheds with uneven bays consisting of simple cladding placed on rafters articulated on one side and sliding on the other.

The spans between the crane gantry girders in these sheds being variable because of ground settlement and lateral displacement, it was necessary to install cranes there with guides on one side in which the rollers pressed against the two sides of the crane rail head.

The Dunkirk steelworks have been in operation since the end of 1962 and we can now give some precise figures for the cost of construction and some details of the operation and maintenance of the plant.

The cost of the foundations for a heavy structure in box construction, including the sand piles around and in the caisson foundations is 20.4 per cent of the total cost of foundations plus unpainted steelwork. If the sand piles are excluded the ratio falls to 11.62 per cent.

The ratio of cost of foundations for a light shed constructed with I-sections to the total cost of foundations plus unpainted steelwork is 9.35 per cent without piles but including compaction by rolling.

The cranes with guides on one side behave well under certain conditions. The horizontal forces measured are less than those stipulated in the German Standard DIN 120 for cranes with rigid carriages perfectly square and robust, but slightly more than those in DIN 120 when the carriages are articulated and more flexible.

The paints with a zinc content of more than 90 per cent, which were applied to a surface of 1,500,000 sq. m. (16,137,000 sq. ft.), have behaved remarkably well as they still correspond with Scale 9 three years after application, and after two years of use over a surface representing 98 per cent of the total surface treated.

In areas where we knew the pH value would be acid or alkaine, the application of one or two extra coats with a rubber-base has given excellent results.

In areas where we subsequently discovered similar pH values, but had used only two coats of zinc paint, the paint has not stood up quite as well.

On the other hand we have been amazed by the resistance of zinc paints to high temperatures.

Finally, no significant corrosion has developed on the unpainted interior faces of the plates in the box portals since the manholes were sealed.

Michel PUECH

Zinc Prevents Corrosion

(Original text: German)

"What would we be without steel?" Paul Valery has written, and Professor Portevin has added "What would steel be without zinc?"

If steel frames are to play a more important role in the construction world, it is essential that they be provided with an efficient and durable form of protection against corrosion.

Zinc supplies the perfect answer to this problem, and can be applied by variable methods, in particular, hot-dip galvanizing, zinc metallization and zinc-rich paints.

Architects, engineers, builders and also users, are taking more and more interest in steel protection, but it is still carried out in a crude fashion and consists of the application of a coat of "anti-rust" paint on top of rust which is already formed or to mill-scale, that is to say, without any preparation of the surface to be protected. It is certain that, in these conditions, no matter what the number of finishing coats of paint ,the protection over a period of time will be illusory and the maintenance aspect important.

In competition with concrete, steel sections offer magnificent possibilities in the sphere of the prefabrication of sub-assemblies for subsequent assembly on site by bolting or, more frequently, arc-welding.

In the meantime, in order to compete on an equal footing, the durability of steel must be guaranteed.

Hot-dip galvanizing, a practice more than 100 years old, is applicable to all the current hot-rolled sections used in constructional steelwork without risk of affecting the basic mechanical characteristics, including the new E.C.S.C. sections with thin flanges.

It can be equally well applied without difficulty to the high-reistance steels designed for the manufacture of cables and also used in pre-stressing.

Allied techniques, of which the principal is the arc-welding of galvanized steels, have permitted the extension of the protection afforded by zinc to constructional steelwork in general, and the instances of its application occur at an increasing frequency:

After Air France at Orly, the swimming-pool at the French Stadium at Boulogne-Billancourt, the Exhibition grounds at Nancy, the Exhibition grounds at Mulhouse, the U.D.K.Garages at the Dunkirk integrated steelworks, steel greenhouses for agriculture and horticulture, and a programme of municipal swimming pools and abattoirs.

Allied Techniques

Arc-welding

This is carried out in the usual way, employing an electrode sheathed in rutile-cellulose, or CO_2 gas with consumable wire, or a conducting flux, or by spot welding.

The welding of galvanized steels was, until recently, completely out of the question, and even forbidden by certain authorities. Now, on the completion of investigations carried out by our organization in conjunction with the Institute of Welding we can formally state:

that arc-welding will not alter the mechanical characteristics of the weld-metal or the heat-affected zone;

that the zone affected by the welding operation, that is to say the area where the zinc has volatilized under the arc temperature, can be easily touched up by

(a) the application of a zinc-rich paint (95% pure zinc minimum content),

(b) metallization.

This method of welding can be equally well applied to metallized steels

Painting of galvanized surfaces

To meet the demands of appearance, different qualities of paint have been developed which can be applied directly to galvanized surfaces. In no instance do they necessitate preliminary preparation of the galvanized surface, which was the case previously and which was effected by the application of a coat of "wash-primer". In this case, therefore, only the top coat needs maintenance, there being no need to maintain the hot-dip galvanizing.

Conclusion

Starting with simple sections, galvanized by mass-production methods, it is therefore now possible to fabricate sub-assemblies in the shops for subsequent assembly on the site, that is to say, using all the characteristics of prefabrication in volume, with a form of protection which is complete, efficient and durable, since it requires no maintenance.

Robert GABRIEL,

Future Steel-Framed Construction (Self-Contained Skyscraper City)

(Original text: German)

In my contribution to the discussion I should like to deal with some town-planning problems, from the architect's viewpoint, in connection with the theme "Future steel-framed construction".

The rapid and unceasing "horizontal" expansion of towns and cities requires more and more building land and the sacrifice of areas of meadowland and forest. Eminent town-planners foresee serious difficulties in the future and seek new solutions, for the time is bound to come when no more building land will be available. With the expansion of the towns, traffic problems arise, which cannot be solved — even at considerable expense-by the construction alone of elevated roads. The increasingly great distances between people's homes and their places of work compel them to acquire means of transport and give rise to traffic congestion, especially at rush hours and peak business hours. Additional factors are the lack of parking space, air pollution by exhaust gases and dust, noise nuisance, the time spent in travelling increasingly longer distances and the resulting strain on the nerves and health of the community at large.

The attempt to find a solution by establishing satellite towns with their multi-storey blocks of flats leads to increased isolation and loneliness for the occupants, since the life force of the town is lacking, whilst traffic conditions have been aggravated rather than relieved.

The inferences to be drawn from those facts have led me to design the "self-contained skyscraper city" which I should now like to introduce to you.

The scheme embodies a steel-framed structure with a height of 4100 ft. (1250 m.) and a diameter of 210 ft. (64 m.). These dimensions emerged of necessity, as I wanted to give the occupants the benefit of the life force of a town in addition to the pleasure of living in rural surroundings. Residential accommodation and the place of work are incorporated in the design for the "self-contained skyscraper city". Within the "city", traffic facilities will be provided by adequate high-speed lifts and paternosters which involve the minimum amount of travelling time, with the least possible amount of noise and no air pollution. The 8,000 dwellings — each with an area of about 1,075 ft.^{2} (100 m.^{2}) — embodied in the scheme provide accomodation for some 25,000 people. This corresponds to the population of an average small town. Taking into account the anticipated numbers of visitors from outside, the approximately 7.5 million ft.^{2} (700,000 m.^{2}) of business and cultural accomodation provide adequate places of work for the occupants.

The four-storey basement of 985 ft. (300 m.) diameter contains parking accomodation for 4,000 cars, while the eleven storeys below it contain storerooms and cold storage chambers for consumer goods. An atom-bomb-proof shelter at a depth of nearly 200 ft. (60 m.) is also provided.

A town with the same number of inhabitants would, as hitherto laid out, require some 500 acres (200 hectares) of building land, *i.e.*, about 21.5 million ft.^{2} (2 million m.^{2}).

Except for the plan area of 325,000 ft.^{2} (30,000 m.^{2}) occupied by the "skyscraper city", the countryside and woods are preserved in their natural state. The access roads are routed underground into the foundations already at a distance of a third of a mile (500 m.) from the skyscraper.

Many problems which have hitherto appeared insoluble can be solved with the "skyscraper city".

Some additional advantages and possibilities offered by the scheme are as follows :

(1) The dwelling shares in the life force of the town, thus obviating the risk of isolation for the occupants.

(2) An area of about 500 acres (200 hectares) is not built on; woods and meadows are preserved and can still be utilized for forestry and agricultural purposes.

(3) In wet weather the upper limit of the clouds is usually at an altitude of about 2,000 ft. (600 m.); above that there is sunshine (stratus clouds at greater altitudes are of no significance).

(4) Unlike conventional towns, the "skyscraper city" cannot go on growing in size indefinitely and thus upset all trafic calculations. For structural reasons it cannot become any larger, so that the traffic calculations initially completed will always remain correct.

(5) Radio and television transmitter equipment providing reception over a very large area can be installed at an altitude of 4,100 ft. (1,250 m.).

(6) The height of the building offers many possibilities for meteorological measurements.

(7) In the large sanatorium which will be accomodated in the top part of the building, allergic conditions such as whooping-cough, hay fever, etc. can be speedily cured. (For sufferers from heart diseases another sanatorium is provided in the lower part of the building).

(8) The dust limit is reached at about 1,000 ft. (300 m.); above this, the air is free from dust.

(9) In the residential and business part of the "skyscraper city" traffic accidents caused by motor cars will be entirely eliminated. Accidents with lifts hardly over occur.

(10)The circular structure encloses the maximum volume for the minimum external surface area, so that the amount of heating required will only be about one-sixth of that required in buildings of conventional design.

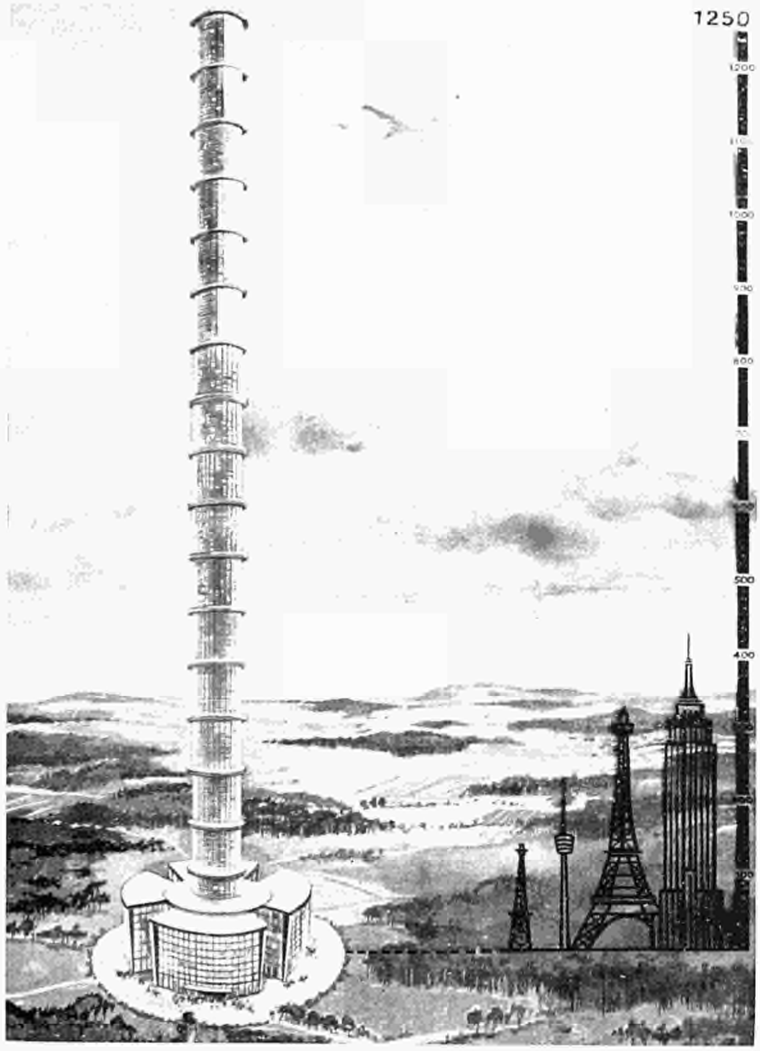
(11)At altitudes above abt. 1,500 ft. (450 m.) wind-generated electric power becomes a paying proposition. In the part of the skyscraper above that level, wind-driven generating plants with a total capacity of 40 MW will be installed. Power supply is safeguarded by a thermal power plant. In addition, an attempt will be made to work an inter-connected power system with the nearest power station.

(12)The service life of conventional buildings (about 100 years) will be far surpassed. The specified corrosion-preventive treatment of the structural components justifies the assumption that, with the exception of wearing parts such as stairs, etc., the building will last indefinitely. Below ground level, steel components will be encased in concrete as protection against corrosion.

(13)The anticipated 10,000 visitors per day, the necessary facilities for their reception, the sanatorium, the research laboratories, all provide employment for a large number of people, for whom the 8,000 dwellings are kept available. In addition, because of the numerous visitors, a permanent exhibition of home and foreign products can suitably be included, and the staff, representatives, and others associated with this exhibition will also requine residential accomodation.

(14)In the top part of the building the occupants will have the benefit of "Alpine sun" without the objectionable accompanying phenomenon of heat usually associated with artificial sunlight.

This scheme will require about 500,000 tons of steel. If the idea of the "self-contained skyscraper city" materialises and proves satisfactory, the use of steel in urban building construction would be substantially increased.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Henri BOLLAND

The Contribution Made by the Preflex System to the Development of Multi-Storey Buildings

(Original text: French)

Current European development in the construction of multi-storey buildings is dominated by three characteristic tendencies, the most marked of which is undoubtedly the increase in height.

The vogue for building multi-storey blocks has been brought about not only by the high cost of land in cities, but also by the fact that the cost of construction itself is reduced by industrialized building. Human and functional factors also play their part as large buildings facilitate contact between individuals, services and communities. They also increase general well-being by improving living and working conditions. From the town planning point-of-view, the concentration of dwellings and working areas reduces the cost of the infrastructure, highways, sewers, electricity, gas and water services. Larger areas can then be reserved for parks and recreation grounds.

A second tendency is a direct corollary of the first. Large buildings, generally in use as offices, attract each day a considerable number of people from the outskirts of the town who increasingly use their own cars instead of public transport. In order to garage these cars, vast areas have to be allocated for parking, mostly in the basements, which also house the archives, the central heating plant and other services. The multiple uses of the space below ground necessitate the reduction of the number of columns to a minimum, as columns generally impede the circulation of cars and the installation of services.

A third tendency, and the most important, is the architect's desire to have complete freedom to divide up the space on all floors, something which can only be done by the exclusion of internal columns. This demand is the result of the growth and development of scientific methods of administration and site layout.

The need for a free space is no less strict in industrial buildings where automation, machine tools and other developments in production engineering demand a flexibility in arrangement which only the absence of internal columns can guarantee. In buildings destined for education or entertainment or for medical services, similar reasons lead to the same conclusions.

A technique which was created with this object in mind and which, in the past thirteen years has resulted in a large number of varied examples in all spheres of building is the Preflex system, invented by M. A. Lipski, a Brussels consulting engineer. The system is now being applied in many European countries and elsewhere.

What is a Preflex beam?

The Preflex beam is a beam in high tensile steel of quality 52 which is subjected to prestressing, during which time the forces are so applied that the maximum working stress, equal to 0.8 of the yield stress, is reached or exceeded at every point. During this prestressing operation, the flange in tension is encased in concrete. When this has reached high strength, the prestressing forces are released, the stresses in the steel diminish, while the concrete is heavily compressed. In service, the stresses in the steel increase again without exceeding the stresses reached during prestressing and the stresses in the concrete diminish until they reach zero, thus guaranteeing that no cracks appear in the concrete encasement.

The following problems are solved by this new technique :

(1) The elimination of internal columns necessarily leads to very large spans, varying from 10 to 30 m. Compared with traditional construction, the strength of the beams must be considerably increased. The preflexion technique has solved this question by permitting the rational and economic exploitation of high yield stress steels.

(2) Correspondingly, the head room for large span beams has to be of the same order as for beams of traditional length. With prestressing, the depth of section can be reduced to 1/40 of the span. It is therefore possible to

achieve the maximum number of storeys for a given height.

(3) With steel beams used in the normal manner, such results would not be possible without a simultaneous increase in the stiffness of the beams. This result has been achieved with the Preflex beam by encasement with concrete which will not crack, precompressed in the area of the tensile flange of the steel section. The method also permits the elimination of permanent deformation due to the residual stresses in rolled products.

(4) It is often necessary to avoid successive deformations which can appear during construction and result in imperfections in both the framework and the cladding. The use of beams in which preflexion has been introduced in the work partly resolves this problem. Preflexion on the site, as in the Office National de l'Emploi and the Tour du Midi in Brussels, provide a complete answer.

(5) The construction of huge buildings necessitates the investment of vast sums of money which must be rendered productive as early as possible. The use of preflexed beams reduces construction time by eliminating shuttering, by providing easy connection between steel and concrete columns and other prefabricated elements, by the reduction in depth of foundations, by using the beams for the strutting of the basements during construction and by their ease in handling.

Fabricators of structural steelwork will appreciate the fact that the preflexion technique has introduced steelwork into many types of structures where the use of conventional rolled sections would have been prohibitive.

Thousands of preflexed beams, already in use, show that steel, is the most suitable material judiciously employed, and on account of its flexibility of application and its numerous qualities which the Preflex system has exploited to the full.

Description of photographs

1 — Grand Magasin de la Bourse at Ixelles-Brussels. First multi-storey department store with clear spans of 20 m. (66-ft.)

2 — Car park in Brussels comprising four floors each of 4,000 sq.m. (1 acre) with spans of 19.20 m. (63-ft.) In 1956, this building, complete in every way, cost 1270 Belgian francs per sq.m. (17/- per sq. ft.) The depth/span ratio of the beams is 1/29.

3 — Telex building in Brussels, with 19 m. (59-ft.) beams each comprising 13 openings, 320 mm. (12 1/2 in.) in dia. for air conditioning. Depth/span ratio - 1/26.

4 — Institut de Physique at Ghent University. 13 storey building. Depth/span ratio 1/25.

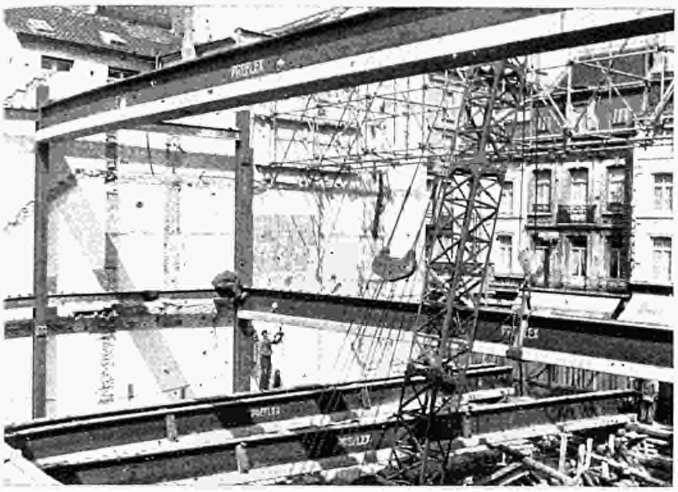
5 — Volkswagen-Porsche Works in Bussels. 20 m. (66-ft.) beams over the workshops.

6 — Eurochemic — Centre d'Etudes Nucléaires de Mol. 16 m. (52-ft.) Preflex beams, having a depth/span ratio of 1/30 which were hoisted with the sliding shuttering also served to strut this shuttering.

7 — Administrative block in Brussels. 21 m. (69-ft.) Preflex beams covering the conference rooms in the basement and strutting the retaining walls during construction

8 — Tour du Midi in Brussels. 40 m. (131-ft.) Preflex beam. preflexed in situ.

9 — Ecole Polytechnique Supérieure in The Hague.



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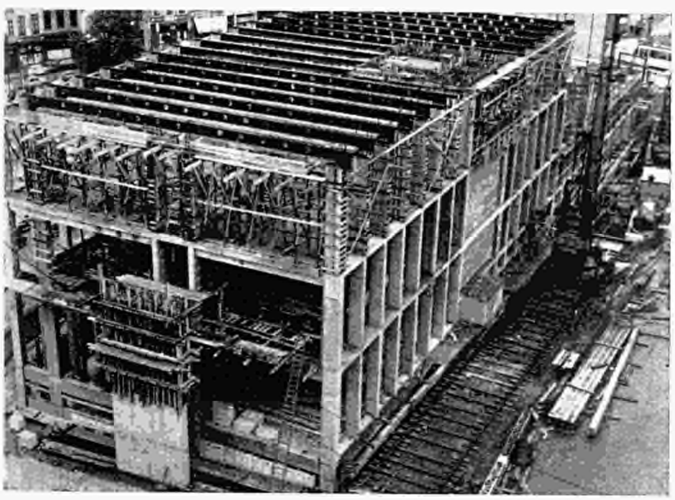

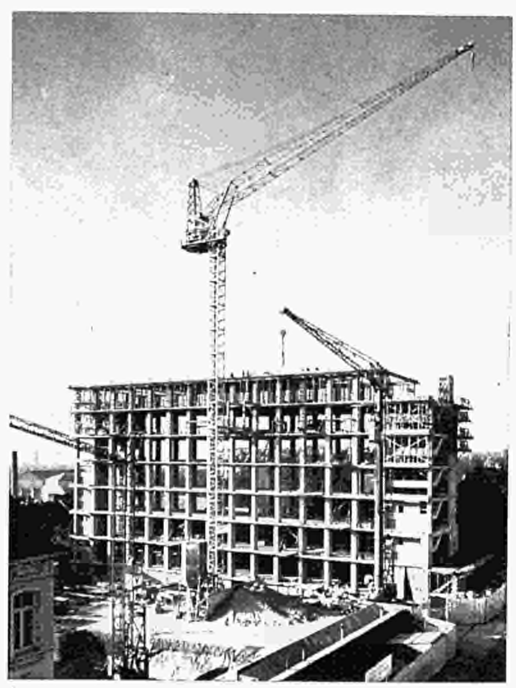

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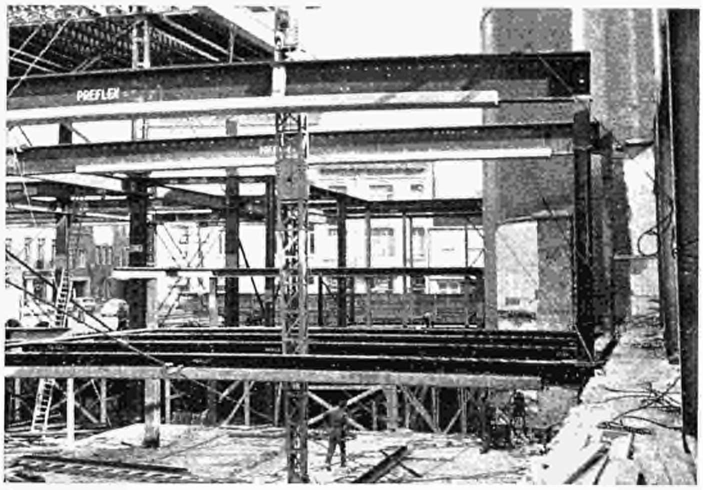

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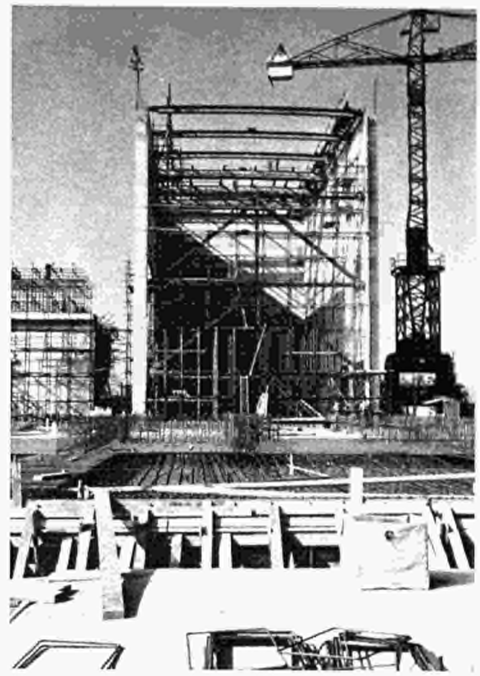

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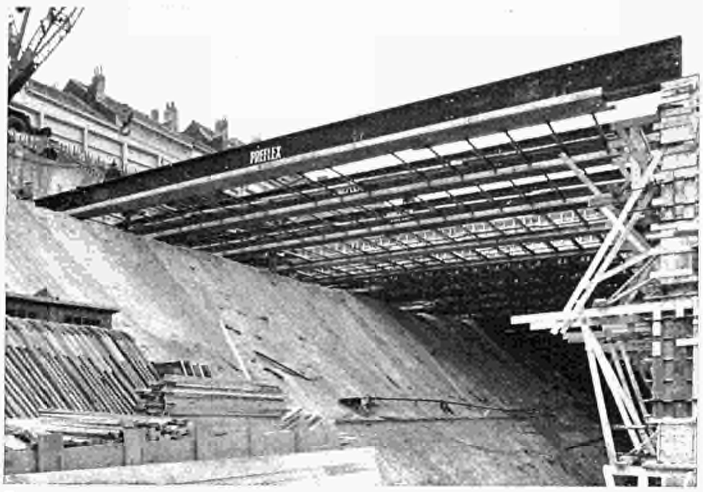

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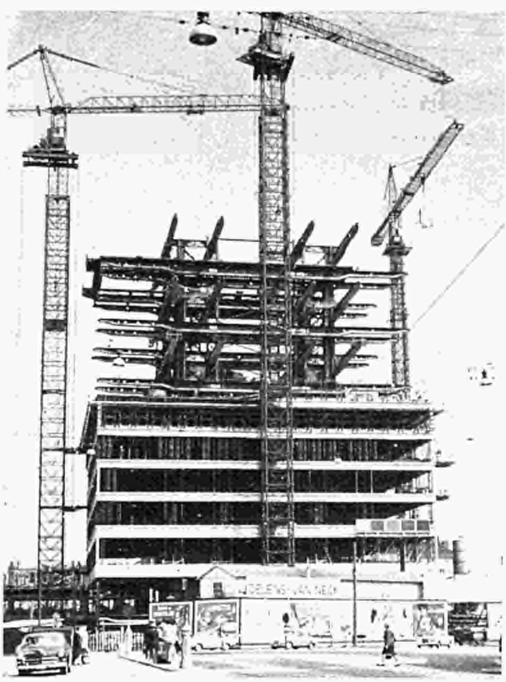

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9

Stanislaw BRYL

Composite Action of Shaped Steel Plate/Concrete Slabs in Building Floors

(Original text: German)

The industrialized erection of multi-storey buildings is leading increasingly to the use of steel plate flooring, whether it be in the form of permanent shuttering or in the form of a structural element. There has recently been a noticeable tendency to take advantage of the natural bond between the steel plate and the concrete slab and to calculate this as a composite structure.

On this basis a series of tests have been carried out in Switzerland for steelwork contractor Geilinger & Co. by the Eidg. Materialprüfungsanstalt (the Federal Material Testing Institute), and I would like to give a summary of the results of these investigations.

For the experimental steel plate/concrete slabs the cheapest corrugated plate and concrete of grade PC 300 were used. In order to filter out the effects of various influences and to obtain statistically worthwhile results, 42 test pieces in all with dimensions 900×300 or 2000×650 were tested (1 and 2). The evaluation of the test results led to the following conclusions :

(1) In the slabs tested, the condition of the surfaces of the corrugated plate (black, galvanised or rusted) had no noticeable effect on the ultimate load; neither did the 400-cycle temperature change from 0 C to 40 C.

(2) The failure of a corrugated sheet/concrete slab is caused by shear or by bending depending on the ratio between shear and bending moment.

Case 1 : Large shear forces — small bending moments, that is the theoretical cracking load is greater than the ultimate load dependent on shear stresses (fig. 1). The section acts as a homogeneous composite section up to failure, which results from the bond strength between corrugated sheet and concrete being exceeded.

Case 2 : Small shear forces — large bending moments, that is the cracking load is smaller than the ultimate load dependant on shear stresses. The section works homogeneously up to the cracking load (fig. 2). The exceeding of the tensile bending strength of the concrete leads to the formation of a crack and to a sudden increase in shear stress. This new shear stress generally exceeds the ascertained failure stress in the slab, which was subject to artificial cracks in the first place. Therefore in this case the cracking load must be considered to be the ultimate load.

(3) The statistical evaluation of the test results (fig. 3) gives a very high variation coefficient of 22% which corresponds roughly to timber construction.

The hypothesis of equal safety for this slab as for composite beams in buildings gives an overall factor of safety of 2.33. The permissible shear stress based on the developed area of the plate is 0.5 kg/cm² and the permissible tensile bending stress in the underside of the concrete section is

$$1.5\sqrt{\beta_d^{28}}\ \text{kg/cm}^2,$$

where β_d^{28} is the compressive cube strength of the concrete.

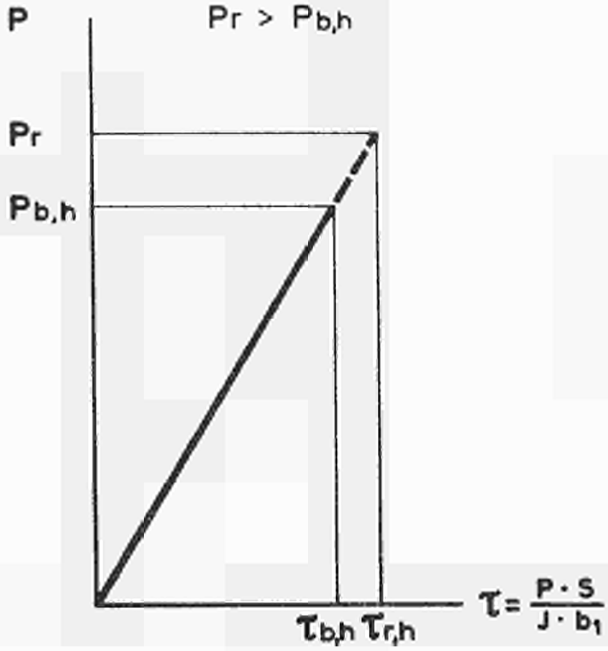

Fig. 1

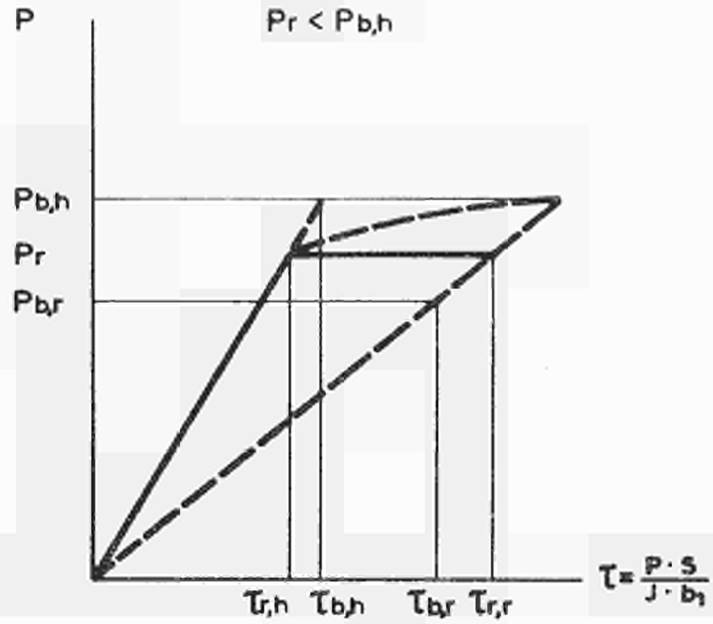

Fig. 2

(4) The steel plate/concrete slab which is constructed without additional shear connectors should be calculated as a homogeneous section provided that the shear and bending stresses do not exceed the permissible values.
Although the proposed permissible values seem at first sight to be very low, the range of application of the composite slabs is comparatively large. For example, for a slab 10 cm thick on a corrugated sheet 100 × 50 × 0.9, the concrete having a compressive cube strength of 300 kg/cm², the uniformly distributed load for spans 1 m. to 2 m. would be 2540 to 635 kg/m² (fig. 4).

Unfortunately time does not allow me the possibility of commenting in more detail on the tests and the proposed method of calculation. I hope however that even this very short summary will contribute to the increasing use of these cheap composite steel floor slabs.

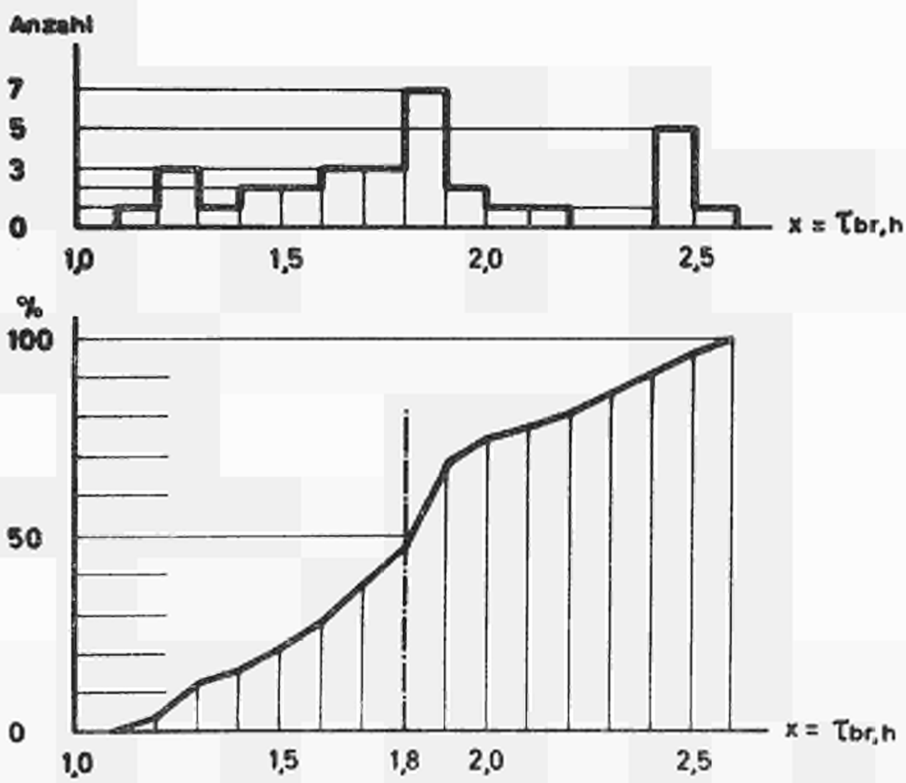

Fig. 3

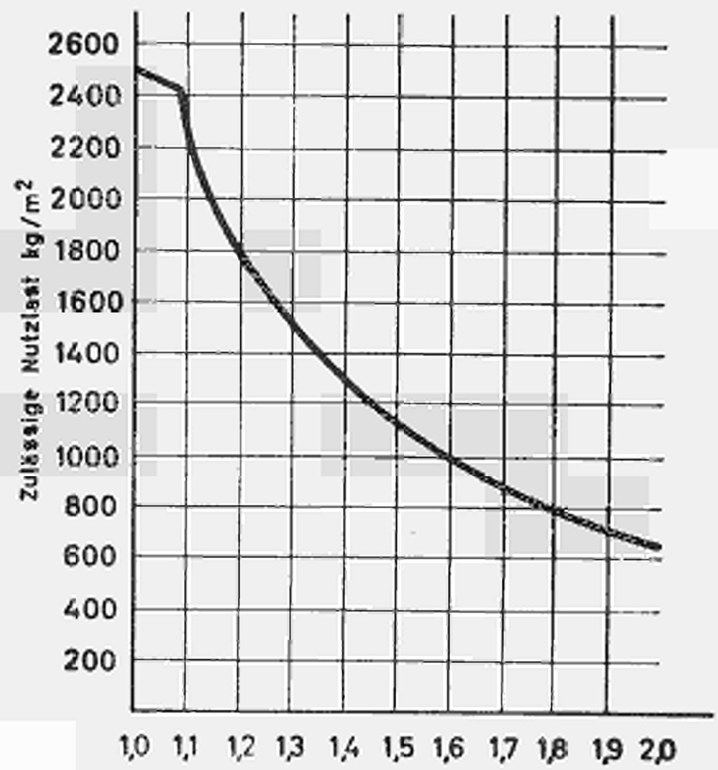
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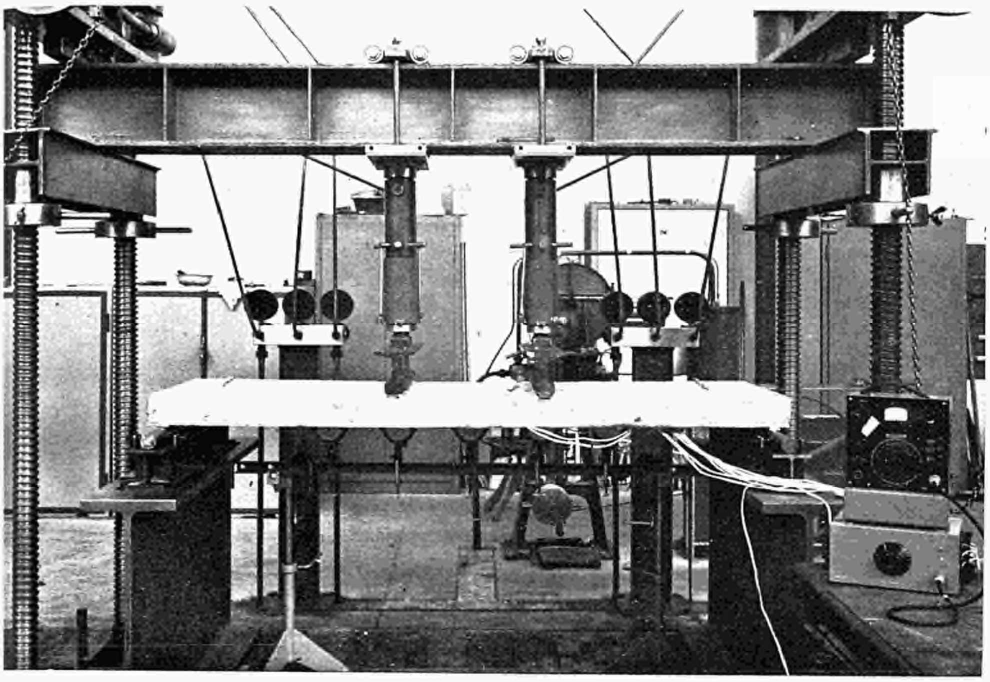
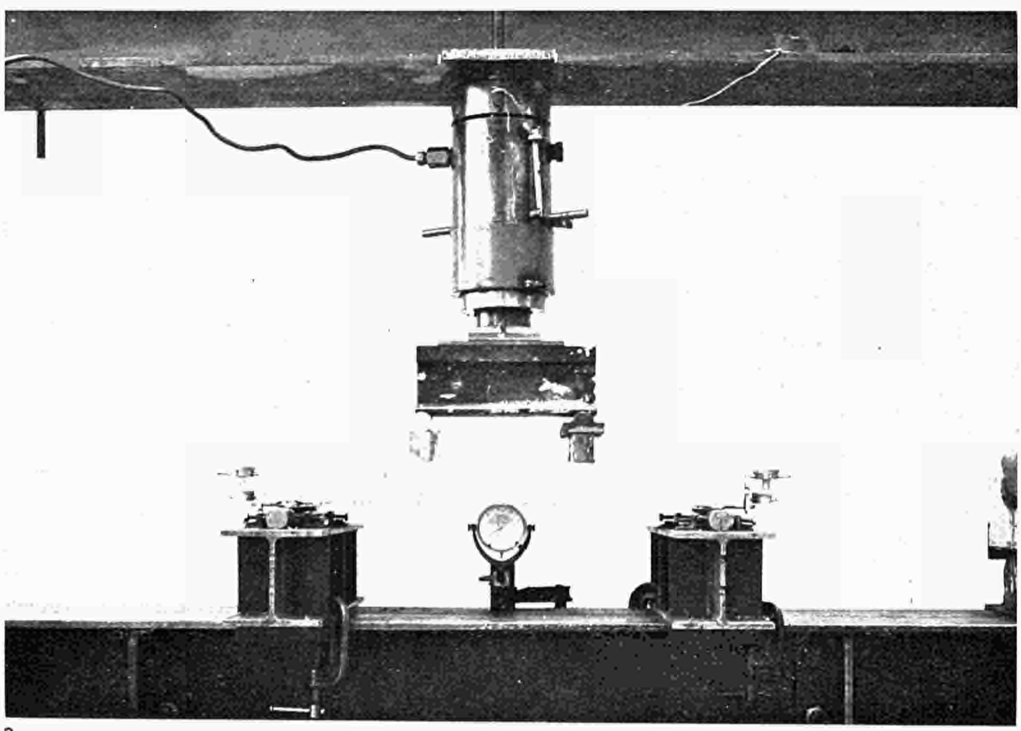
Fig. 4





2

Jean L. SARF

Structural Steelwork and the Aesthetic

(Original text: French)

In 1953, after previous research, it was decided to use steel tubes for the first time as structural elements in the construction of a multi-storey building.

The structure, known merely as the "building in the Rue Jouffroy", (1) was the prototype for a whole series of buildings which have since been erected.

Among the buildings in this series must be mentioned the first skyscraper in Paris, the apartment block in the Rue Croulebarbe, (2 and 3) in which the logical complement to the use of tubes is introduced. Tubes filled with concrete are utilised as structural elements, the two materials acting compositely.

At that same time, similar structures made their appearance in other countries. Such buildings were erected, for example, in Düsseldorf in Germany for Mannesmann and Phoenix-Rheinrohr and in Milan, Rome and Turin in Italy.

The utilization of this structural element in conjunction with other standard sections shows, that the steel tube can be used wherever appropriate, either in large or restricted amounts, in the same way as other sections *e.g.* the building for Air-France at Orly.

The idea which served as the base for the earlier buildings was extented in these structures, (4 and 5) the aim being to reunite the aesthetic with the rational by using a steel structure to its greatest extent. Thus, by a simple presentation, devoid of all frills, a new architecture appeared.

This was possible because the architect, Edouard Albert, knew how to extract the maximum effect from all the constructional elements in the structure and was able to show the possibilities that steel can offer in modern architecture.

The same aesthetic ideas in the conception of the useful and the rational can be seen in the hall, 55 m. (180 ft.) in clear span, at the Marché-Gare in Toulouse (6) where the traditional riveted structure, used nevertheless with particular care, does not in any way destroy the form of the building.

Once the field of tubular steel structures had been opened up, it was interesting to progress into industrial buildings which, although utilitarian by definition, can also present an agreeable appearance.

In the second part of the Massey-Fergusson factory in Beauvais (Architect M. Gridaine), we believe we have shown that this is quite possible and within a price range comparable with traditional construction (7, 8 and 9).

In the second part of the Massey-Fergusson factory in Beauvais (Architect M. Gridaine), we believe we have shown that this is quite possible and within a price range comparable with traditional construction (7, 8 and 9).

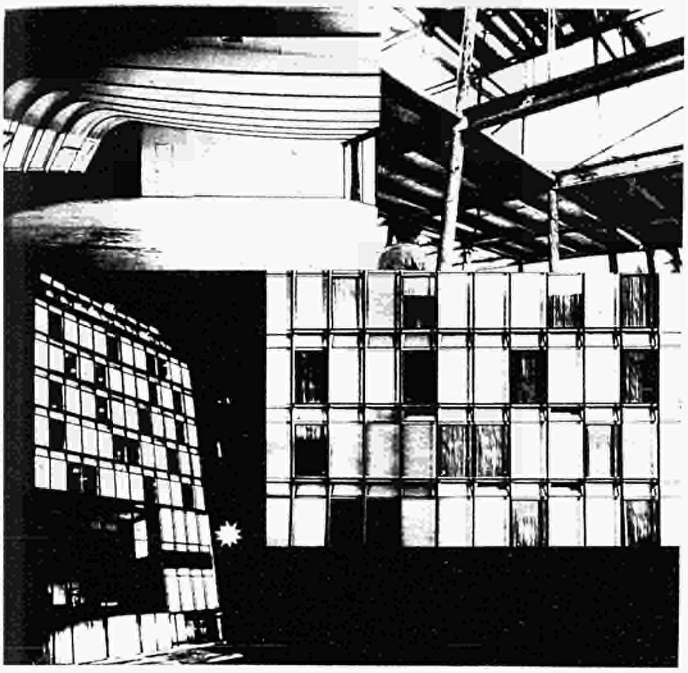
The same idea may be seen once again in the construction of a depot for Philips-France at Toulouse (10 and 11).

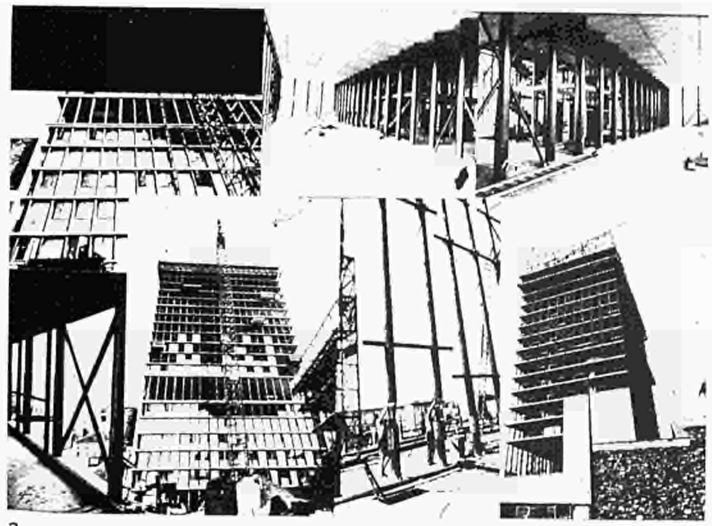
Lastly, the building at Perpignan, constructed for S.C.C.A.A.A., has shown that a structure comprising rolled sections and tubes, designed with a care for aesthetics, can meet with the approval of all (12).

As long as designers do not lose sight of aesthetic requirements, I, for my part, forecast that the future for structural steelwork is wide open and competitive with other materials, even its direct rivals, reinforced and prestressed concrete.

Description of photographs

1 — Apartment building at Rue Jouffroy, Paris (18e).
2-3 — Apartment building, Rue Coulebarbe, Paris (13e).
4-5 — Administration Building of Air France, at Orly.
6 — Marché-Gare, Toulouse.
7-9 — Massey-Fergusson factory building, Beauvais (Oise).
10-11 — Depot for Philips-France, Toulouse.
12 — S.C.C.A.A.A. building at Perpignan.





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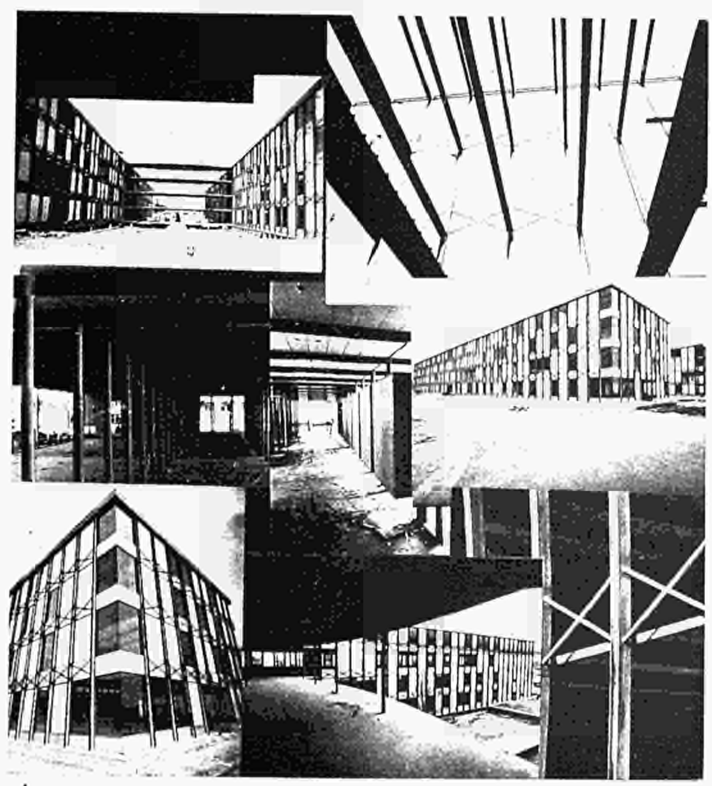

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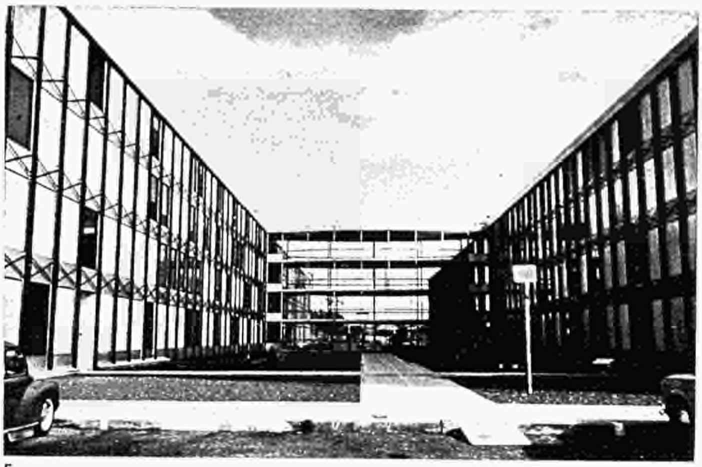
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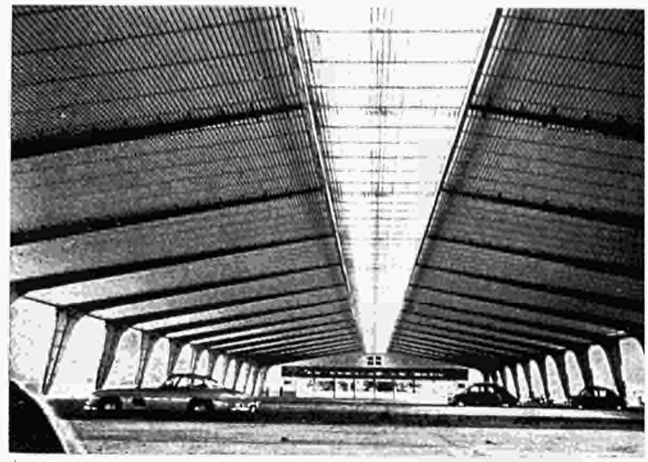
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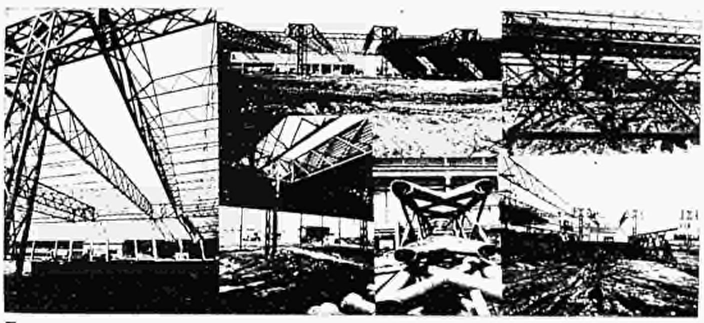
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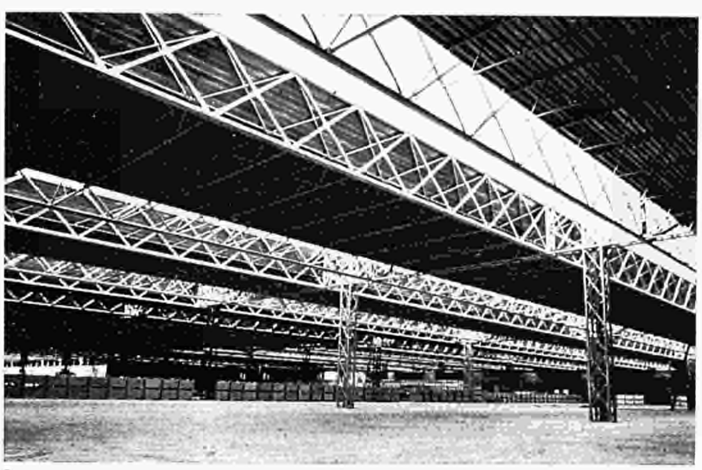
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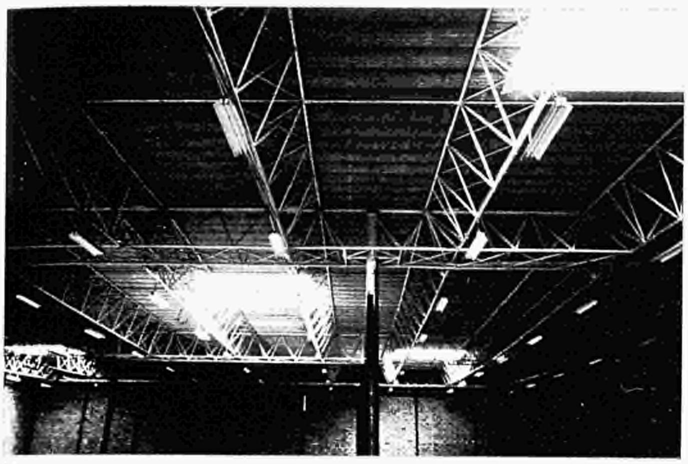
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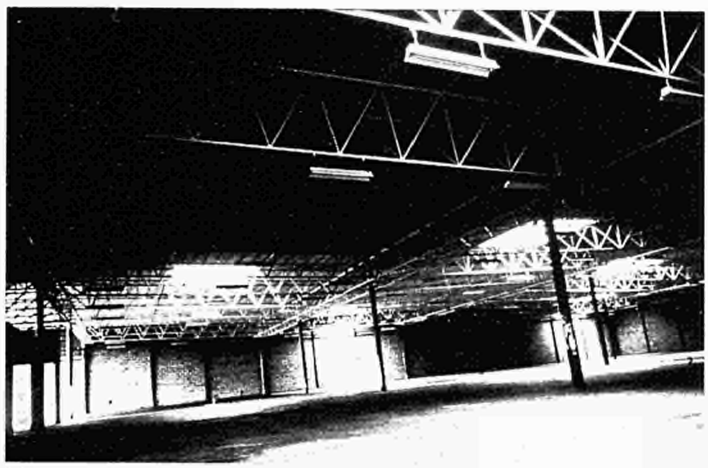
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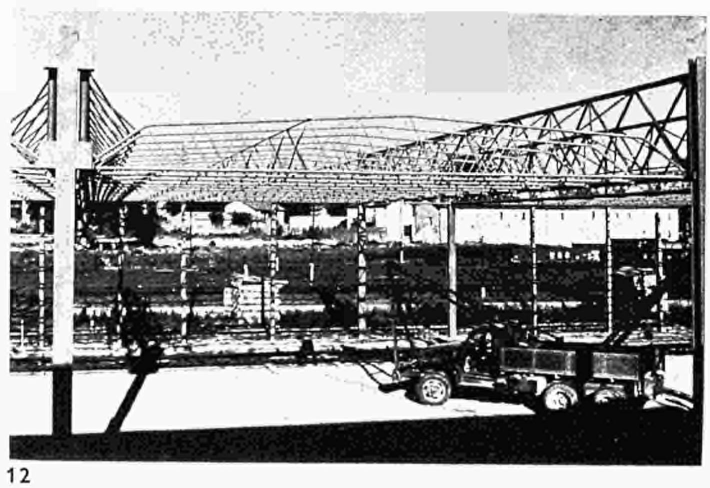
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L.A. ASHTON

Research on Fire Protection in the United Kingdom

(Original text: English)

There has been reference in two of the papers presented to the Congress to the problem of fire protection in relation to structural steel, and it is therefore opportune to describe investigations now being made in the United Kingdom. Some background information is necessary for an understanding of the purpose of the investigations.

Countries which include in their building codes or regulations requirements for fire protection, specify for different classes of building the fire resistance to be assigned to each part of the structure contributing either to the stability of the building or to confining a fire to as small an area as possible. The fire resistance of an element of structure is given as a time, 1/2 hour, 2 hours etc., for which a similar part fulfils certain stringent requirements in a laboratory test. There is broad agreement in the various national standards for fire resistance tests on the heating and loading conditions, and on the criteria of performance. There is also general acceptance of the assumed equivalence between test duration and actual fire severity in a building expressed in terms of the calorific value of the combustible material per sq. unit of floor area expected to be present (fire load). It is customary to quote the fire load for a building as the weight of wood in kg/in² of equivalent calorific value to the contents.

The fire resistance specified for a given building is likely to vary from one country to another, since fire protection is a complex subject containing legal, technical, policy and even emotional aspects. Where policy has limited objectives for fire protection, say only safety of life, it would be reasonable to expect less stringent requirements for fire resistance than where policy seeks to reduce property loss or damage to the structure.

Although there might be agreement between the testing laboratories in different countries on the amount of given types of insulation necessary to enable structural steel members to achieve the various periods of fire resistance policy could well be an overriding factor in determining the use of the test data.

The technical aspects, however, being matters of fact are more readily dealt with, and it should be possible to establish whether or not the doubts about the validity of the present assumptions of the equivalence of test duration ande fir severity are well founded.

Research on a small scale has been in progress for some time at the Fire Research Station to determine the factors affecting fire severity in compartments of buildings of relatively low fire load. The British Iron and Steel Federation is now co-operating to extend the investigation to full scale with particular reference to structural steel. The research has shown that fire severity cannot be properly expressed without taking account of the air supply (ventilation) and a better measure than the amount combustible material per unit area of floor would be given by the amount per unit area of window.

The co-operative investigation will be made in a building erected for the purpose which will permit experiments with fire loads varying from 7.5 to 60 kg/in² as timber cribs, with two different levels of ventilation. Protected and unprotected steel columns and beams of different sections will be located in pro-determined positions within the building and externally close to the windows and their temperatures will be measured throughout each experiment. Similar members will be exposed to heat in the same manner in the standard fire test, and a relationship obtained between test and actual fire severity.

As an introduction to the main programme a test was made in a building simulating a modern flat constructed with a steel frame. The test was completely realistic, all details of construction being reproduced, and the contents and furnishings being typical for this type of occupancy. Columns and beams were encased with 1/2 in plasterboard, an amount of protection which is less than that permitted at present in the U.K. for high flats. A burn-out of the contents (average fire load 25 kg/in²) was allowed to take place, but none of the columns reached temperatures at which their strength would be significantly affected. Rather high temperatures were measured on certain beams but not such as to be critical for their stability.

Diulio Sfintesco

(Original text: French)

May I make one or two points which Prof. Zeevaert's account of earthquake-proof or, as he calls them, "earthquake" structures brought to mind. It was stated that metal construction, the art of building in metal, was the art of designing and effecting connections. This is undoubtedly a fact. But some doubt has been cast on the behaviour of welded connections in structures which have to stand up to earthquakes.

I should like to ask two questions. First, does this scepticism arise from practical observations? Have instances of unsatisfactory behaviour of welded connections actually been recorded ?

Secondly, we are making more and more use of high-strength bolting for connections in this type of building. Is there any factual evidence as to the behaviour of these connections in buildings which have been affected by earthquakes?

Leonardo Zeevaert

(Original text: English)

The **first question** is: why I do recommend for construction of steel structures the prefabrication of welded sections and joints in the shop, and subsequent riveting of connections during the erection of the structure in the field, instead of recommending an all welded steel structure.

The decision to follow the plan of design and construction mentioned above, is a difficult question to answer if it has to be generalized, because it depends on environmental means of construction as: size of milled steel sections in the region; costs of an all riveted structure versus welding; workers to do welding; good supervision and control of welded connections in the field. I would not have serious objections to make welded connections in the field, where there is a good workmanship and control; however, if there is the slightest doubt of obtaining good welding during erection; it is better to use riveting because it is safe and may be easily performed and controlled.

Fifteen years ago I designed in Mexico City, a 12 storey building of all-welded construction, that is to say, welded sections and joints were prefabricated in the shop and, during the erection, the connections were welded in the field. The result was that in order to obtain a good structure, a tremendous amount of skill, control and supervision of workmanship in the field were required, to be able to get good welded connections and avoid defective welds and high distortions during welding. However, it appears to me that n the United States of America and here in Europe, methods of welding in the field have developed to such an extent that one is able to obtain reliable welded connections in the field, under difficult conditions. Therefore, my above statement does not apply to those countries where an advanced state of welded steel construction in the field has been achieved; nor does it apply in the case of very light welded connections, where welding in the field can be performed in favorable conditions and may be easily supervised and controlled.

The **second question** was on the use of high working stresses in steel structures for seismic design, and the meaning of flexibility in building structures. Concerning seismic design in which high elastic stresses in the steel have been allowed to take place, or where stresses have been even permitted to reach the plastic range during ground motions, in order to obtain maximum economy in the design; I may say that this way of thinking depends on the type of structure and costly architectural materials that have been used in the project. The factor of safety for different buildings and structures should also be considered, since the behaviour of different types of buildings, and their functions may imply different safety factors; for example, a Warehouse could be designed with less factor of safety than a School, a Hospital or a Hotel. Therefore, when one talks about using high stresses during strong ground motions, one should consider the final behaviour or damage that may be allowed in the proposed structure.

In any event, it is desirable to learn about the response spectrum during the earthquake and the maximum forces that may take place; the fundamental period of vibration of the structure should be designed away from the dominant period of the ground; if this may be achieved one could use during ground motion stresses in the steel up to 90% of the yield limit. Because of ductility in steel, if these stresses during a strong ground motion, reach the plastic range, much energy may be absorbed and if the individual structural elements are well designed as explained in my paper, there is no danger of collapse. However, it is not sufficient to design the isolated structural members close to the elastic limit. One should consider as extremely important that the moment resistant joints will not be overstressed, as they may loose their strength and rigidity in greater measure after each earthquake if they are permitted to work in the plastic range; most important in these cases are riveted connections.

Concerning flexibility there are two schools of thought. The first one allows the structure to be very flexible, permitting large distorsions and also considering that a great amount of energy during the strong ground motion will be absorbed by the architectural interiors, as walls and other facings that could wedge against the structure and therefore are able to absorb energy. This means that during the earthquake, the frame although working in the plastic range, will not collapse, but the interiors might be damaged beyond repair. This way of design is a costly proposition in regions where earthquakes are very frequent, as after each earthquake the buildings thus designed, have to be repaired in their costly architectural details; expensive curtain walls of stone break and distort; partition walls, door frames, etc., are easily damaged.

The second procedure of design, used also in several countries, is that of producing a very rigid structure, the period of vibration of which is usually below the dominant period of the ground. The structural frame is designed in such a way as to take entirely all the earthquake forces and large distorsions are not permitted, as the deformations of the structural frame are restricted by means of heavy concrete walls. If the structure obtained is very rigid, it may be stated in general, that it will take higher forces in comparison with a flexible structure, more so if this building is founded in firm or moderately firm ground. The result will be a heavy and expensive building in contrast with a flexible structure designed in the plastic range. As regards the two ways of thinking in the design of structures mentioned above, I may say that I do not agree with either one of these procedures.

Based on my professional practice I recommend the design of a structure and building using the concept of "controlled flexibility", high stresses may be allowed during ground motions, but never exceeding the elastic limit. The earthquake force shall be taken by the structure frame. The architectural elements forming the building should be carefully considered during seismic motion. By this, I mean that flexibility in the building and stresses should be controlled to be compatible with: (a). The response spectrum, and therefore the dominant period of the ground, to avoid resonance and hence the magnification of stresses. (b). The structure could be designed with high working stresses; but in such a way that it will still work in quasi-elastic conditions during the strongest unusual aerthquakes. (c). All architectural details should be carefully designed for earthquakes, permitting them to work in compatibility with the distorsions of the steel structure; these elements should be fixed to the structure in such a way that they will be capable of taking the earthquake forces induced in the buildings. Expensive facing should be carefully considered and designed to avoid damage.

A building with steel structure and steel secondary elements, like curtain walls and partitions, designed with controlled flexibility as the buildings shown in the paper, should fulfil their functions with little or no damage in the architectural details and may be easily and unexpensively repaired after a major earthquake has taken place.

In case of the unhappy event that the building structure should be subjected to an earthquake much larger than the strongest foreseen in the design of the building, this structure will have reserved strength when stresses reach the plastic range, to survive the earthquake without collapse and with moderate damage to the architectural details.

Findings

The findings to emerge from the proceedings of Working Party III can be summarized as follows :

(1) It is desirable that steel construction should turn more and more to new structural forms which can better utilize, and take advantage of, the intrinsic qualities of steel and of the possibilities presented by the development of the techniques for putting them into practice. This kind of trend is already manifesting itself in space-frame structures and in the various new forms of wide-span roof.

(2) The use of new types of thin-walled components, particularly those made of normal bending quality sheet and tubes, offers promising possibilities in this connection.

(3) As regards assembly, automatic welding with proper checking of resulting seams is calculated to produce first-class jobs at comparatively low cost. More use should be made of high-strength bolts.

(4) As an experimental basis for scaling down the requirements demanded of steel construction, a study might be made of the resistance of structural steel frameworks to fire by organizing fire tests for complete frameworks, as nothing short of this can give an idea of the overall impact of fire. Experiments of this kind have been carried out in Switzerland, in Germany, and more recently also in Britain.

(5) The problem of corrosion has been successfully coped with for all degrees of exposure. Reference was made in particular to good results obtained by metal cladding and by coating with paint having a high zinc content.

(6) Experimentation is felt to be needed to measure the effective strains produced by wind in the columns of multi-storey buildings, as the safety margin afforded by the present regulations may well be excessive. Studies should also be conducted on the behaviour of metal structures in response to dynamic impulses, such as, in particular, earthquake shocks.

(7) A study of the combined use of steel and concrete in a single structure, more especially of the composite type, is regarded as indispensable to the future of construction work. It is suggested that a symposium of steel and concrete technicians might be organized on the subject.

WORKING PARTY IV:

Prefabrication of Steel Building Components

Chairman:

Ir. A. VAN AALST

Rapporteurs:

Lucien WAHL

Dr. - Ing. Otto JUNGBLUTH

The Working Party discussed prefabrication of steel building components, *i.e.* their mass production in the factory workshop, away from weather and other on-site hazards.

It was considered that in view of manpower difficulties and rising labour costs, and also of competition from other materials, such as concrete and timber, industrialization, and hence mass prefabrication of steel components or complete structures, was absolutely essential.

Prefabrication, however, involved a great many problems, which must be studied and overcome by co-operation among all those concerned, the constructional engineering firms, the architects and the clients.

For prefabrication to be introduced, the following points must be effectively dealt with: development of prefabricated steel components for use on their own or in conjunction with other materials; length of production runs expected and hence amount of capital expenditure required; adaptability of components for different purposes, so as not to restrict freedom of architectural design; standardization; local and national building regulations; arrangements for transporting prefabricated components.

It was clear from the proceedings that architects were keenly interested in the use of prefabricated steel components, and anxious to establish constructive contact on the subject with the builders.

Lucien WAHL

Prefabrication of Steel Building Components

(Original text: French)

In approaching building problems with a view to prefabrication, one needs continually to keep in mind the population increase of the post-war years. For the E.C.S.C. countries alone Mr. Van Ettinger gives 50 million as the number of housing units that will have to be built between now and the end of the century.

This population explosion brings with it immense demands for buildings of all types; residential housing of course, but also for schools, hospitals, offices, factories, and so on. Our civilization owes itself the duty of finding means, commensurate with the needs, of building a lot, quickly and at acceptable cost.

Industry alone can cope with this demand, by virtue of its methods, which aim to replace manual labour by machinery wherever possible, both on the building site and in the factory, also by virtue of its attitude toward research, work organization and similar matters.

> "Full-scale industrial methods must be applied to building and introduce standardization of housing components.
>
> What is needed is to create mass-production-mindedness, mass-habitation-mindedness, and mass-building-mindedness"

wrote Le Corbusier in 1920.

Forty years later, these words still hold good in their entirety, for, although industry is taking an ever-increasing part in construction work, the progress made by the building trades has been slow. Considerable advances are still possible and these words, which in their day might have seemed utopian, have today a prophetic ring.

Our work here is devoted to the study of prefabrication of building components in steel, meaning their fabrication in the workshop, sheltered from the vagaries of the weather and the hazards of the site. The part played by industry has so far made only a slight impression, but one can discern a change in which steel construction can and must play a major rôle.

We shall first look into the meaning of the word "prefabrication" and some of the conditions that have to be present in order that greater use may be made of prefabricated components in building work. Secondly we shall take a look at the various components that are suitable for prefabrication and finally touch on some fields in the use of prefabrication.

Let us then commence by discussing **prefabrication** and all the conditions that have to be present if greater use is to be made of prefabricated components in building.

A number of misconceptions repeatedly occur around the term "prefabrication". This is because the buiding industry, more often influenced by the preoccupation with immediate profit than by architectural considerations, has all too often put on the market buildings whose appearance, to put it mildly, left much to be desired, also because the expression "prefabricated construction" has, in the minds of some clients become synonymous with ugliness, inconvenience and makeshift. There is no reason why this should always be so, and we shall certainly have an opportunity, in the course of this session, of seeing examples which will help to alter this opinion.

Another difficulty concerns the very conception of such building. A building - say a one-family house - is a complex thing. Its construction involves the work of many different trades, one dovetailing into another. The manufacturer will try to reduce to the minimum the number of different trades engaged on the site and to fabricate complete assemblies in the workshop. Nevertheless, by this off-site fabrication he takes the risk of limiting architectural freedom and the system that he involves will become less and less simple. The "closed" system of prefabrication at which he arrives entails fabricating the whole structure beforehand in the workshop - or at least all the main structure - and offering the building to the client, out of the catalogue as it were.

This practice, tempting as it may well be, is liable to end in partial failure. This is the lesson to be learned from the experience in other countries; for one cannot industrialize the site, the approaches, the inlets and outlets for services, the climate, or the landscape.

Local and national regulations offerd extra restrictions. Personal preference, reaction against uniformity also come into the picture. In the end, the market that the manufacturer thought was a very big one boils down to a few hundred units, or at most to about a thousand a year.

This is considerable but it forms an insufficient basis for full-scale mass production.

The prospects for the "open" system of prefabrication are quite different. This means shop fabrication of the functional components of the building; components that can be used and embodied in buildings by many users and put together by those in charge of the work in many different ways to produce the whole projects.

Here the quantities required can be on a different scale altogether, in the region of at least ten thousand, no longer a mere hundred. Fully industrialized production methods can be envisaged.

To achieve this end howerer, these prefabricated functional components must fit easily together and be interchangeable. They will be at the architect's disposal as with any other building components.

This all seems obvious enough. One might even ask why this method of assembling prefabricated units has not been adopted commercially on as wide a scale as might have been expected. There are many reasons, but an important one is that, up to the present no dimensional rules have existed to tell one party the dimensions to which he should fabricate and the other the dimensions to which he should design.

These rules are now in process of being established, and this is the purpose in France of the standard specification AFNORP 01 101, dated July, 1964, and, at international level, of the work of certain committees of the European Productivity Association and of the International Centre for Building Documentation. To put it very briefly, this French standard fixes a basic module of 10 cm. (3 15/16") with "multi-modules" or preferred dimensions of 30 cm. and 60 cm. (11 13/16" and 1'11 5/8") for horizontal dimensions and 20 cm (7 7/8") for vertical dimensions between floor levels. It also prescribes how lengths should be measured (between wall faces, centre to centre of columns, as the case may be) and shows the methods of measuring clearances and tolerances! Let us hope that in European practice common rules for dimensioning will receive ever wider acceptance, in order to provide the basic conditions for the big market necessary to full-scale industrialization.

There are other factors favouring a wider use of prefabricated components, namely:

— rising labour costs which oblige contractors to be constantly on the outlook for ways of reducing the number of man-hours required for the erection of a building, and, above all, for ways of reducing the time spent on site,
— mechanization, automation of the shop work,
— transport facilities.

In this already wide market with immense growth potential, we are convinced that steel and steel construction have a decisive and major rôle to play.

If all the advantages of prefabrication are to be fully effective, it will be necessary to fabricate components in very well equipped workshops. They will then have to be transported, often a long way from where they are fabricated. They must therefore be comparatively light though still able to comply with many other requirements.

In choosing the basic material, account must be taken of mechanical properties, weight and price. Steel, more than any other material, is favourably placed in this connection.

Furthermore, the techniques of fabricating steel products have passed all tests and ensure the precision so necessary when prefabricated components are to be used. This means that it must be possible to assemble the parts on site without the need of fitting adjustments. In structural steelwork shops the parts are machined to a tolerance of about 0.040" (1 mm.), sometimes less, and this degree of precision is adequate as a tolerance for infilling units - external walling panels - curtain walling units, which are supported by the steel framework. This degree of precision is the more readily attained when the production run is long enough to justify extensive, and consequently, expensive tooling that will ensure the requisite precision.

We now come to the second part of this talk, dealing with the various kinds of components that lend themselves to prefabrication; we shall quickly run over the various items that make up a building in order to discover those to which prefabrication techniques can usefully be applied.

Steel framework for buildings

To start with the skeleton - when a building is to be steelframed, the framework is always fabricated in the shops prior to being assembled on site. Structural steelwork engineers, who have always followed this practice, must think that when we talk about prefabrication, we are beginning to discover something that has been a habit with them for years and this is in part true!

Nevertheless, in structural-steel fabricating shops, real mass production is rare and the fabricated components vary for every building that is erected. Given sufficiently long runs of one piece, several hundred and not a few dozen, it should be possible to install the equipment required for each operation, to introduce new automated plant, to use certain welding methods as yet little employed in our industry and, lastly, to reduce the costs of fabrication, while at the same time ensuring a higher degree of precision. Prefabrication is not only a question of fabricating in the shops, but also of mass-production, entailing all the changes in techniques which the introduction of industrialized methods implies.

We agree then that the framework for buildings is always to some extent prefabricated, without necessaily conforming to the conditions of fully-industrialized fabrication. The necessity of abiding by certain standards, the fixing of storey hights, for example, in blocks of flats limits the number of sizes in which columns have to be fabricated and conduces to the adoption of longer production runs and perhaps specialization by certain shops which will acquire the necessary plant. The same applies to the beams of the structure, the framing for floors and roofs.

One type of structure is particularly suitable for prefabrication, namely the three-dimensional or space-frame structures, made up of steel members selected from a small range of sections and connected to the nodes by a variety of proprietary methods. These members form a sort of network which, while being very light, can span great distances. Systems like these have usually been built of tubular steel sections. It is not impossible that we shall see the development of structural systems in which the framework for the horizontal planes will be constructed in this fashion.

Continuous horizontal components, floors and roofs

A great number of floor systems based in varying degrees on steel construction have been devised. In France however, such applications are rare and in the field of housing almost non-existent. In low-rental flats, where the need is greatest, the floors are nearly always made up of a solid reinforced-concrete slab, the undersurface being merely painted, thus saving the cost of a false ceiling.

Steel is employed more generally:

— for the floors of industrial buildings

— in the case of office buildings and residential blocks where the floor joists may be left exposed (rarely found) or where the concrete floor is poured on a ribbed steel sheet that serves as permanent shuttering and is left visible.

— Finaltily where the floor has to accomodate a great number of services, as in the case of air-conditioned office blocks, and where the use of a false ceiling is justified. Various systems of steel flooring are particularly suitable in such instances.

The use of ribbed steel sheeting forming shuttering for the pouring of concrete floor slabs is to a certain extent an example of prefabrication. But ultimately we might have a floor the component parts of which would be prepared entirely in the factory and would reach the site finished, complete with false ceilings containing acoustic insulation, electric wiring, appropriate floor finishes, etc. These units would then be joined together to supply all the floors for the building.

Steel is being widely used in roofing, either in the form of galvanized sheets or for constructing troughing, which, with or without a weathertight covering, makes up a substantial portion of the roofs of industrial buildings. Here we have prefabrication in which a few manufacturers have laid down certain sizes.

Continuous external vertical components, walls

For these, light-weight walling, external wall panels and curtain walling are being used on an ever-increasing scale. This trend will probably become still more widespread by reason of the standardization drive in progress pretty well everywhere. it may be worth-while here to enumerate very briefly the main advantages of light-weight external walling:

— light weight, giving a wall with thermal and sound insulation characteristics equal to those of external walls in conventional materials, but having something like one-tenth of their weight.

— The saving in weight obtained by the use of light-weight walling makes it possible to have lighter framework and smaller foundations, and consequently reduces costs in respect of these items. This is particularly the case with tall buildings, where the reduction in the sizes of sections for the structure yields a useful floor area which is so much the greater when the structure is in steel;

— the reduced space taken up by the structure, resulting in a larger useable floor area within the building;

— speed and ease of erection, without the need for ultra-powerful handling and lifting appliances; hence a saving in man-hours on site.

Methods of prefabricating heavy items can also be economically applied to the construction of external walls, with less site labour, but they are hardly suitable for small-scale housing schemes and for sites far removed from the fabricating shops. Light-weight external walls on the other hand are easy to transport over long distances and can be used even for small-scale building projects.

They will be used more and more as local wage rates rise and standardized dimensions become more widely known and accepted.

If we are to enjoy all the advantages that accrue from the use of light-weight cladding in building, the finish must be without fault. It might perhaps also be necessary for a large part of the first and second fixings to be made an integral part of this type of walling and for it to include in its construction, skirtings, electrical and heating installations and the like. This is possible only where these walling units are so designed that they can easily be incorporated into the construction of buildings with many and varying plans.

It is therefore essential for very close co-operation to exist from the outset between the manufacturer and the architect so that walling units of this kind will combine flexibility of layout with uniform external appearance. This problem is difficult, but perhaps not altogether insoluble.

It is also essential that the state of the market should admit of large production runs so as to offset design and tooling costs.

Light-weight external walling can be made of a number of materials, but steel appears to be the best choice, on account of its mechanical properties which provide the requisite strength in spite of its light weight, on account of the precision with which it can be worked, and on account of the various finishes that can be applied to it, *viz.* stainless steel sheating, zinc coating, paint, enamel, plastic sheating. Finally, steel can be used in conjunction with other materials, such as glass of different types, reconstructed stone, etc., in which case it ensures the mechanical precision of the panels and provides an easy method of fixing them.

Vertical components, partitions

Internal partitions in residential buildings are for the most part built of conventional materials and so entail a great deal of work on site. Also, in large-scale housing schemes, they are often made up of heavy prefabricated panels or rammed concrete. But such a method of construction, though admittedly cheap, definitely lacks flexibility and the house once built cannot be altered.

On the other hand, it has been found useful to be able to alter the internal layout of offices at will, and so movable partitions are gaining in popularity. Steel has proved to be the ideal material for this purpose.

I am convinced of the advantage there would be in changing the layout of a house or flat in step with family changes. Perhaps in the future, when these needs have been recognized and the problems of lighting, heating, etc. have been resolved (as they have already been in the case of office premises) people will be prepared to pay the extra cost such schemes entail. The developer will offer a fairly large floor area with few internal supports. The purchaser will arrange the internal layout to suit his immediate requirements and will be able later to rearrange it easily. A steel structure, which requires only a few supports and lends itself to wide spans, is ideal for carrying out such an idea. Piped services will be laid in the floor in steel panels; the partitions will be movable and convertible

Other components

Many other components can be prefabricated in steel, but we shall enumerate only a few here:

— stairs can be in steel;

— metal windows; these are in everyday use both in modified conventional construction and in many prefabricated systems. In France, about one quarter of the total demand for windows is met by steel products;

— door and window frames in steel are also in common use:

— finally, we have heating installations, electric wiring and service pipes, sanitary and domestic fittings.

Having listed the components that can be prefabricated in steel, I should like to start on the third part of my talk and take a quick look at several forms of construction, discussing briefly a few cases where prefabrication is practised.

In France at any rate, **industrial buildings** account for two-thirds of the work of constructional steelwork engineers and so one might expect to find a particularly intensive standardization drive in this field. In most European countries some firms and groups of firms have standardized their products and put on the market buildings of a particular type of standardized dimensions. For example, one such groupin France isadvertising standard buildings of from 2 m. to 12 m. (6' 6 3/4" to 39' 4 1/2") in span with heights of from 4 m. to 12 m. to 10 m (13' 1 1/2" to 39' 9 1/2").

The British Government, under its decentralization and regional development policy, has worked out and produced designs for standardized pre-built factory premises, which are made available to manufacturers ready for occupation. A similar development is taking place in other countries.

Dr. Jungbluth will tell us, tomorrow, I believe, of some moves towards standardization in Germany. Generally speaking, however, it is surprising to find that the engineers start again at the beginning each time they have a building to design and go through all the calculations again, as they have to construct a building with a span of 49 ft. (15 m.) or one with a 50 ft. (15,27m) span.

Each time the parts have to be marked out afresh in the workshop and the machines reset, so that the structure must eventually carry all these on-costs which amount to about 15%, at least, of total building costs. Standardization would bring about a marked reduction in cost. This is not a thing we are used to, but it is nevertheless possible and could come to be adopted on a wide scale provided the standardized structures were sold at prices well below those of structures designed and built as individual "one-off-the-line" jobs.

Now it is clear that the great bulk of industrial building will never be standardized. This sector belongs to private industry in whose internal affairs the State plays but little part, so that its advocacy of standardization cannot make as much impression as it can elsewhere. Moreover, requirements in this sector vary widely and the constructional steelwork industry is particularly organized to meet the varying demands of industrialists.

There is, however, one sector in which the construction and erection of buildings on mass-producion lines has met with particular success, in France at any rate. I refer to the ready-made steel-framed shed-type farm buildings which are being sold from the catalogue. Several structural engineering firms specialize in this type of building. Their selling methods enable them to keep their fabricating shops working to capacity, making the finished parts for stock during the winter months for a market which is essentially seasonal. This gives food for thought.

School Buildings. This is another sector of building in which steel has already secured a certain foothold. The demands of education are very pressing and steel makes for great speed in building.

Furthermore, the layout of a classroom can be fairly rigidly fixed and adopted for the whole country; standardization is a fairly easy matter here. Lastly, and this condition is essential for the development of building methods requiring enormous investments in brains and material, often out of all proportion to the size of the firms concerned, schoolbuildings belong to a sector in which the State has power to impose certain regulations or to assume responsibility for the costs involved in design.

It is not for me to describe here the many systems of fabrication that have been developed for the construction of schools. I have personally received information about methods developed in France and on ten or so others designed elsewhere, some of them in England.

All are notable for their adoption of a module (1.75 m. i.e., 5' 9" for school buildings in France, recently increased to 1.80 m or 5' 11"), the use of a simple steel framework for supporting floors, walls, light-weight walling and partitions and, more particularly, a system of prefabrication more or less "open", according to the system followed.

The building of hospitals also offers a field which favours. overall standardization with an extensive use of prefabricated parts, but the design work here is less advanced than in the case of school buildings.

Housing. But the prime necessity lies in the sphere of housing, a category in which the use of prefabricated steel units can and ought to progress.

For one-family houses numerous prefabrication processes have been worked out by steel engineers in all countries. Yet the saving in building costs are not sufficient to overcome in a decisive fashion the prejudices and habits of individuals who often think mainly in terms of brick and stone. A change in this mentality is gradually appearing and economic conditions should enable prefabricated houses to find a real market.

Several systems were on display at the Industrialized Building Systems and Components Exhibition held at the Crystal Palace, London in June 1964. In France, the Scientific and Technical Building Centre (C.S.T.B.) has for its part approved several types of private houses built of units involving steel construction. Other systems of construction are being worked out in Western Germany, in other E.C.S.C. countries and elsewhere.

Of all these systems, the only ones with a future will be those which combine real aesthetic merit with great adaptability and flexibility thereby ensuring that neither architects nor private clients will be prevented from developing original ideas of their own.

"Low-rise" blocks of flats of one to four storeys in height constitute the major sector with prospects of a steady increase in the use of prefabricated building components. Here the hight is not such as to require heavy steel framework, which would call for ultra-powerful handling equipment. This is, moreover, a sector where the size of the projects does not always warrant heavy prefabrication, but where rising labour costs on site demand some change.

Many systems were also shown at the Crystal Palace Exhibition and are beginning to be tried out in Great Britain. In this sector of low-rental flats government assistance is very common, and systems of building will develop more effectively as public authorities provide the means for experimentation on an adequate scale.

In the building of blocks of four to five storeys, heavy prefabrication has been adopted and appears to be suitable. Steel components are being used here to varying degrees. The use of prefabricated steel components will expand and further progress may be expected in the years ahead.

For tall buildings, over 165 feet (50m.) in height, every construction job assumes such dimensions that it becomes a law unto itself and justifies the working-out of a complete building system for the purpose with extensive use of prefabricated components. One might go so far as to say that these buildings could not in most cases be built without structural framing in steel and certainly not without the use of prefabrication methods, which, we would repeat, derive from the regular practice of structural steelwork engineers.

To round off this account of present-day application of prefabrication and some foreseeable trends, I should like to return once more to the essential fact that this development will not be possible without the closest possible co-operation between all who take part in the business of building and to conclude by quoting another pioneer of industrialization, Walter Gropius.

> "Genuine variety without monotony could have been produced if architects had been interested in undertaking the basic preliminary studies necessary for prefabricating standardized components of a kind that could be assembled to give a great variety of types of dwelling. The idea of prefabrication has, however, been taken up by firms who have concentrated on the prefabrication of complete houses rather than on the prefabrication of parts of buildings.
>
> The monotony that would result from the adoption of the latter would increase the apprehensions of the public, who for sentimental reasons feel ill at ease in a prefabricated environment. Today, skyscrapers are built of prefabricated components but industrialized residential building is still only in its infancy."

It is very much to be hoped that this symposium organized by the High Authority of E.C.S.C. will result in some progress in this direction.

Otto JUNGBLUTH

Standardized Steel Structural Components and their Manufacture on Automated Production Lines

(Original text: German)

Introduction

On viewing the technical development that has taken place in the last few decades one gets the impression that never before new scientific knowledge has had such an effect upon the human race or more accelerated technical progress than in our present era. It is appropriate therefore that the engineer should for his own narrow range of activity pause from time to time and consider whether the field of engineering that he represents is progressing along with the general technical trend or is lagging behind other technical disciplines.

Looking quickly at some recent scientific and technical achievements, one perceives with admiration and astonishment that it is nowadays possible to produce in an atomic reactor elements which do not occur in a natural state anywhere on earth; that men can travel by rocket at speeds of 19,000 m.p.h. (30,000 km/h); that particles of matter can be accelerated to very near the velocity of light, *i.e.*, 186,000 miles/sec. (300,000 km/sec.); that with 7.7 tons of nuclear fissile material a ship can circumnavigate the earth fifteen times without any additional supply of energy; that medical science, thanks to modern aids, will soon be able to transplant human limbs and organs from one body to another; and that probably in only a few years' time it will be possible to use de-salted sea-water for transforming barren deserts into fertile arable and pasture land. Thanks to present-day automation engineering, increasingly higher outputs together with a reduction in the number of attendant personnel are being attained year by year in the sphere of the mass production of consumer goods. To give but one example: over the last ten years the annual output of motor cars in Western Europe has risen from 1.5 million to 6.8 million units, *i.e.*, an increase of 350 %.

But how do matters stand in the technical progress of building construction and more particularly, what is the position in regard to steel fabrication and its application in the construction industry? Alas, we find that, hitherto, constructional engineering and especially steel construction has remained one of the under-developed fields of engineering and that artisans' methods still largely predominate in the structural use of steel. Yet the transition to industrialisation in the building industry, to mass production, thanks to progressive standardization of types, to assembly-line manufacture of prefabricated components with the aid of automation, is already clearly discernible.

I should now like to describe to you a standardized programme of inter-adapted prefabricated steel components with the associated manufacturing processes for the three sections of constructional engineering, namely, industrial building construction, housing construction and administrative building construction. You will, no doubt, understand that for this purpose I shall chiefly draw upon information provided by the firm with which I myself am connected.

Prefabricated steel roof components

One of the most important load-bearing and space-enclosing parts of a building is the roof. The most commonly employed steel roofs, which I can assume to be sufficiently well known, present undulating or trapezoidal profiles and are mostly constructed from galvanised sheet steel or galvanised cold-rolled strip. A roof design which differs from these well-known forms leads to the steel roof (1) which functions as an orthogonally stiffened plate-type structure. It comprises longitudinal stiffening ribs spaced at intervals of 3 ft. 3½ in. (1 m) and stamped-in transverse stiffening beads at 4.9 in. (12.5 cm) centres. As a result of the widely spaced ribs of high rigidity and the closely spaced beads of low rigidity, suitable load-carrying action in the longitudinal and transverse directions of the roof is obtained.

In conventional roof systems on the other hand the structural members are disposed chiefly in one particular direction only and four or five different components are generally necessary to perform individual special functions. This roof constitutes a single continuous unit, like the stiffened deck plate of a large bridge. This favourable structural behaviour due to the nature of the stiffening employed, which is further enhance by methodical strain hardening obtained in cold-working and which will be more fully considered later on, has made it possible to produce a roof construction with a dead weight of only 2.5 — 3 lb./ft.^{2} (12 — 15 kg/m^2) for unsupported span lengths of up to about 33 ft. (10 m).

During erection, the beaded sheets are connected to the stiffening ribs and to one another by means of a clip-cum-adhesive connection or by cold riveting (Fig. 3). Two-component cold-setting epoxy resins are used as adhesive bonding agents for metal; the sealing compounds employed with cold-riveted joints being single- or two-component preparations. These two force-transmitting, shear-resistant types of structural connection join the beaded sheets and the ribs together into a stiffened elastic continuous whole which in most cases requires no bracing. Tests confirm the "membrane" action of this structural system.

Three possible surface treatments to ensure protection against corrosion are available:

(1) Two-coat lacquering (undercoat + top coat).
(2) Hot galvanising + two-coat finish lacquering.
(3) Hot galvanising + plastic coating.

Plastic-coated steel strip is now commercially available. The P.V.C. coating has a thickness of 200 microns and is one of the most efficient forms of surface protection for steel with regard to weather resistance, wear resistance and resistance to chemical attack. Recently, an American chemical concern has produced polyvinyl fluoride sheeting for application to strip steel, and this material is claimed to have far better corrosion protection properties than the familiar P.V.C.

As the top space-enclosing feature of a building, the roof not only has to combine structurally with the other structural members of the building and to exclude rain, snow and wind, but in many cases it also is required to afford protection against cold and noise. In the case of a steel roof, the thermal insulation, whose correct design is of paramount importance in obviating the risk of dripping moisture due to condensation, can be provided by any of the well-known insulating materials, *e.g.*, mineral wool, cork, fibrous materials and foamed plastics. There are two types of thermal insulation according to its position in relation to the roof covering: a) in the "cold roof", the insulation is suspended some distance below the roof sheeting, the cavity being adequately ventilated by air inlet and outlet apertures. b) The "warm roof" is of single-leaf construction. The essential requirement is a vapour barrier under the insulation; this function is admirably performed by the plastic-coated steel roof with its high resistance to vapour diffusion. The following illustrations show a flat steel roof of this type provided with Plexiglass rooflight domes which can be opened out for ventilation, the opening movement being performed by small electric motors. A significant advantage of the "warm roof" is that with this method of construction it is possible to employ a very advanced degree of prefabrication if assembly-line production is adopted for insulated roof panels as composite units (Fig. 1). These are provided with a beaded steel sheet as the bottom structural member, a plastic-coated steel sheet as the top insulating member, and an intermediate layer of foamed plastic insulating material such as Styropor or polyurethane. During erection the joints are filled with foamed plastic strips and then closed with a sealing compound.

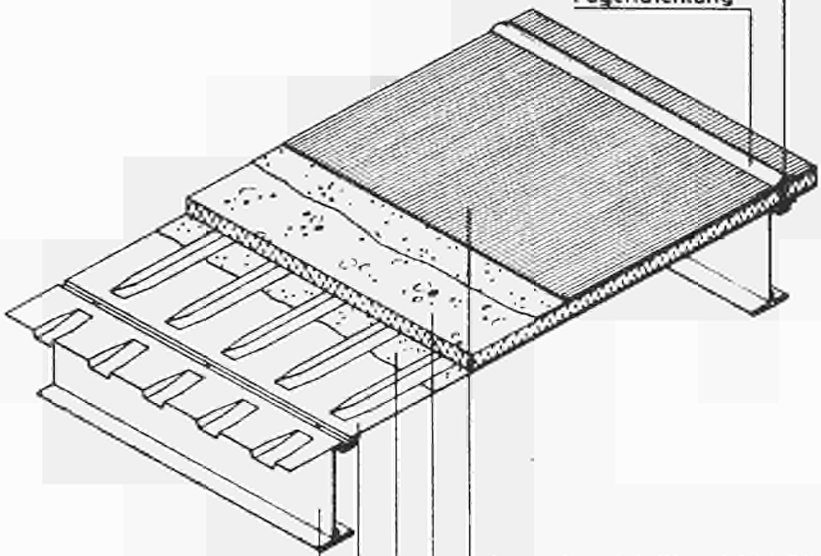


Fig. 1

Prefabricated steel-floor components

The strip-steel floor (2) is constructed on the same structural design principle as the steel roof described above, which functions as an orthogonally stiffened plate-type structure, the only difference being that the sheet employed is thicker and the ribs are spaced closer together. The transversely beaded sheets, which are "cranked" on one edge to accommodate the adjacent sheet and provided with longitudinal and transverse slots, are secured to the longitudinal ribs by means of self-tapping screws. The maximum span of these floors, which have no supporting girders on the underside, is about 33 ft. (10 m), and the maximum superimposed loading that they can carry is about 100 lb./ft.^{2} (500 kg/m^2). The dead weight of the actual steel flooring is extremely low — 6 to 8 lb./ft.^{2} (30 — 40 kg/m^2) — and is advantageous in regard to the structural design of the steel supporting framework of the building. There are various methods of constructing the floor surfacing and the ceiling (Fig. 2), more particularly with regard to the requirements of acoustic insulation. It is hardly necessary to draw the attention to the great advantage of utilising rigid plate-type steel floors as a means of stiffening and stabilising the structural framework. The next illustration shows the strip-steel floor used in the construction of open-plan offices in an administrative building. The dead weight of the floor in this method of construction is a mere fraction of the weight of a comparable concrete floor.

Prefabricated steel-wall components

Prefabricated wall components for residential buildings

Prefabricated wall components can be constructed in a variety of ways (3) and they differ from one another more particularly in regard to the purpose for which they are intended in residential, administrative or industrial building construction. In the case of single- and two-storey residential buildings, the forces due to the external loads are, generally speaking, so small that a structural framework can be dispensed with. If the loads arising from dead weight, snow and wind are transmitted directly into the walls, then, with normal plan dimensions of single- and two-storey housing construction, stiffened wall panels constructed of strip steel with a thickness of between 0.04 and 0.08 in (1 - 2 mm) are easily able to transmit all the forces involved, without the provision of a structural framework of rolled steel sections.

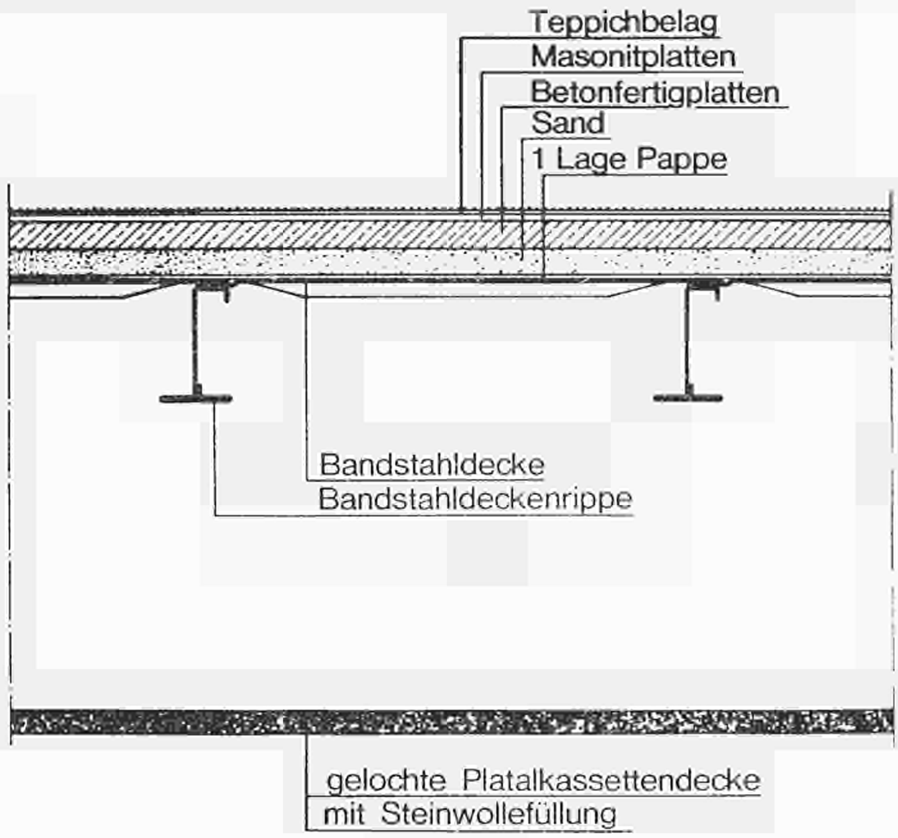


Fig. 2

In contrast with large wall components corresponding in length to a whole room, storey-high wall panels with a unit width of 4 ft. 1 in. (1.25 m.) not only allow of a wide variety of arrangements in conjunction with almost complete freedom of layout on plan, but will also enable the two facing sheets of a panel to be adequately stiffened by means of an edge frame of suitable cross-sectional shape and, which, can be erected without the aid of lifting appliances. Water pipes and electric wiring can be installed in the joints between adjacent wall panels, so that no provision need be made for installing any piping or wiring within the wall components themselves. For the thermal insulation of the wall components and joints the modern foamed plastics are particularly suitable. These materials, even when used in small thicknesses, are fully capable of meeting the insulation requirements of the Central European climate. Whether such wall panels, filled with foamed plastic and faced with steel sheet, are to be regarded as composite members in the sense that the foamed plastic co-operates structurally, calls for further study. Single- and two-storey residential buildings are, of course, not subject to the more stringent requirements of the fire prevention regulations. The mechanical properties however and, more particularly, the relatively low heat resistance of plastics do not, at present, allow of composite structures in which steel and plastics co-operate in the same way as do steel and concrete.

In the case of the wall panels the closed edge frame, in combination with a certain "co-operating width" of the facing sheets, transmits all the vertical and horizontal loads. The foamed plastic filling is utilised as a means of preventing the buckling of the sheets.

A wall construction panel of such design transmits all the loads occurring in single- and two-storey residential building construction, especially if, as in the present case, appropriate structural measures ensure that interconnected panels will act together as a closed "plate". The necessary thermal insulation is also available. The requirements as to corrosion resistance and acoustic insulation must also be fulfilled, however.

Corrosion protection is provided by hot galvanising on both sides of the sheets (zinc coating 25 microns in thickness), together with a 200 microns thick PVC plastic layer on the two outer faces. The thick, protective "skin" of plastic obviates the hard, cold character of the steel facing sheets and makes the walls soft and pleasant to the touch. Since the weight of a wall normally has a significant effect upon the sound-insulating efficiency, such light walls can really be expected to provide a lower standard of acoustic insulation. The fact that the acoustic insulation is nevertheless relatively good is probably due to the multi-layer construction of the wall with its alternation of soft and hard materials and their different kinds of vibration behaviour.

Residential building construction with prefabricated steel components

Prefabricated roof, floor and wall components as described in the foregoing sections may be used for the construction of any form of single-family or two-family flat-type residence normally encountered in practice. Without prejudicing the freedom of choice in determining the layout on plan, there are at present three types available for the detached single-family house (4), with 590, 1,175 and 1,575 ft.^{2} (55, 109 and 146 m^2) of residential space, respectively. The type 146 house is entered through a spacious hall which in turn gives access to the large living room, with an area of 500 ft.^{2} (46 m^2), or to the kitchen directly on the right, or straight on to the study. In front, on the left in the hall, is the door to a lavatory also equipped with a shower bath. The hall gives direct access on the left, to the morning room, which is within easy reach of the kitchen, *i.e.*, the housewife's sphere of activities. The rear part of the hall leads to the bathroom and three bedrooms. The large type 146 bungalow shows how prefabricated steel components are used to build a home. All the walls consist of plastic-coated steel sheet or of partitions in which cupboards are accommodated.

The shortage of suitably developed building land and its high cost, however, necessitate the adoption of forms of residential construction in which the single-family houses are pushed closer together, as in the case of this layout pattern (Fig. 3). Here the houses are built of the same steel prefabricated components as the single-storey and the two-storey "terraced" houses.

Despite all possible thoroughness of planning however, despite the efficiency of technical design and in spite of all the loving care bestowed on constructional detailing, we cannot ultimately evade the question as to whether prefabricated housing construction has a future at all, and whether, more particularly, standardised housing constructed of prefabricated steel components can be economically successful and find its market.

The number of dwellings completed each year in the Federal Republic of Germany has been fairly constant for the past ten years and amounts to a little over half a million. In the countries belonging to the European Economic Community the need for new residential accommodation is estimated as being approximately 1.4 million dwelling units per year over the next few years.

The need is great, but with the present cost of building there are not many who can afford to buy a flat or indeed a house. The motor car, which only a few decades ago was a luxury which only a few wealthy people could afford, has now become a mass-produced consumer commodity available for the whole population. Seventy years ago, when the hand-made Benz-Coupé "Mylord" car equipped with a 9 h.p. engine cost 3,800 Marks, a house could be bought for 15% of the present-day cost. If one ignores the more stringent requirements that have now come to be applied to both product, the price of a car can be said to have remained about the same, whereas that of a house has increased six-fold. Are the high costs of construction due to the steady increase in recent years of building operatives' wages? These wages have not risen more steeply than those of workers in the motor industry. No, the blame lies primarily with the very much lower degree of rationalisation of building technique, which is only now entering the industrialisation stage. If in the future, despite the continually rising level of wages, the owner-occupied dwelling, and indeed the ownee-occupied house, is to become a consumer commodity similar to the motor car — and what reasonable grounds are there for denying this — then the tremendous rationalised reserves of constructional technique will have to be utilised to the full. Prefabricated structural components then will really come rolling off the assembly line, the factory-made house will no longer be a Utopian dream, and correctly designed prefabricated steel components will make their full contribution to the volume of building construction. With present technical resources it is possible to produce prefabricated housing components on largely

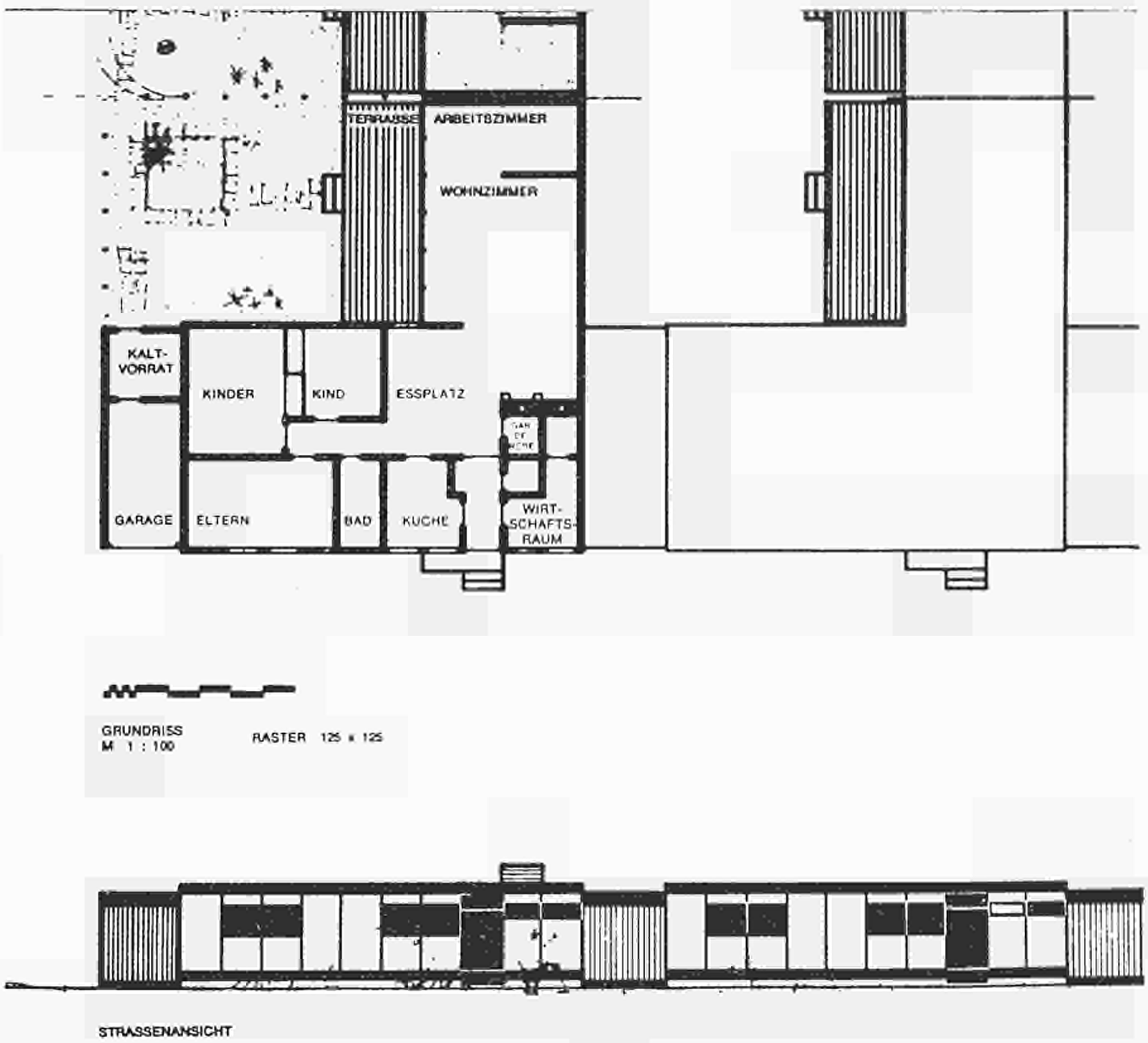


Fig. 3

automated assembly lines at such favourable cost that the price of the house ready for occupation can be in reasonable and appropriate propotrion to an ordinary middle-class income.

With the aid of such prefabricated steel components for roofs, floors and walls the steel construction industry should be able to secure a great share of residential building activities, *i.e.*, be in a position to participate in the future expansion of the market.

Prefabricated wall components for administrative and industrial buildings

Once it is acknowledged that prefabricated construction — and steel construction is prefabricated construction — is the building method best suited to our present-day economic structure, it becomes pointless to provide a steel-framed shed-type industrial building with brick infilling walls. With coated-steel wall panels such buildings can be clad in the shortest possible time even during periods of severe frost. Interlocking steel panels of trapezoidal cross-sectional shape, faced with plastic, (fig. 4), are bolted to horizontal sheeting rails of cold-formed sections, no bolts being left externally visible. Thermal insulation is provided by plastic-lined steel sheets with a foamed plastic facing on one side, bolted internally against the outer leaf of the trapezoidal section . The thermal insulation coefficient of this rapidly assembled prefabricated wall of plastic-faced strip steel is five times as high as that of the half-brick walls which are otherwise most frequently employed in industrial building construction. With this type of prefabricated wall, paintwork is eliminated. The facades of multi-storey administrative buildings (5), are particularly well suited for prefabricated construction, and the past two decades have shown that, because of their low weight and ease of erection, steel, aluminium and glass are much favoured for the purpose.

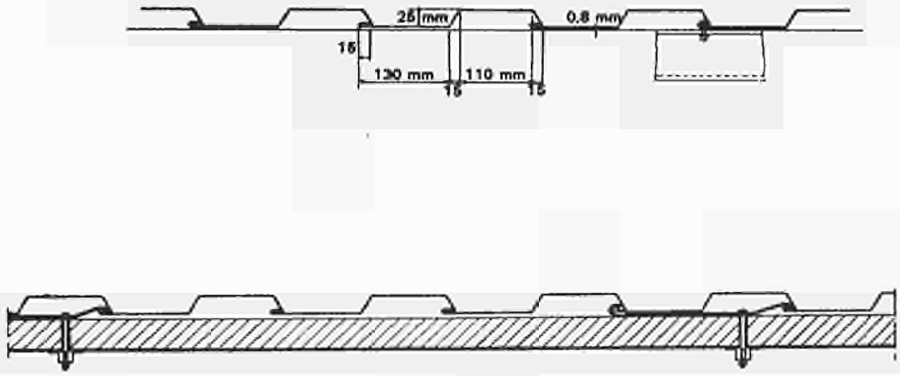


Fig. 4

From the numerous possibilities of using steel sections and sheet-steel panels for cladding units for the façades of buildings I should like to call attention to three characteristic types. In the first example folded steel strips are installed as a "cold" exterior wall, with air cavity, in front of a concrete wall slab. In the second example a grid of steel sections is mounted in or before a steel or concrete structural framework. Windows and insulated spandrel panels are mounted in this "cladding grid". This form of construction is more particularly-referred to as "curtain walling" In this case the outer leaves of the insulated wall panels consist of beaded pre-coated roofing sheets. Finally, the third example shows the use of storey-high insulated structural components, as are also used in housing construction. Because of the greater storey height, these components have, in the present case, been strengthened by means of rectangular tubular sections at the butt joints (6). In all these cases, plastic-faced sheet steel has been employed.

Standardized frames for shed-type industrial buildings

An inquiry into the need for prefabricated shed-type buildings which covered a wide section of industry, showed that the most frequent need is for factory buildings and storage buildings with spans in the range from 33 ft. to 100 ft. (10 - 30 m), with a pronounced maximum at 66 ft. (20 m). A German mining concern has accordingly chosen a series of spans between 41 ft. (12.5 m) and 82 ft. (25 m), (Fig. 5) with increments of 8 ft. 3 in. (2.50 m), for its programme of standard portal frames for shed-type buildings — both for those with and for those without overhead travelling cranes. The standardization of the prefabricated components enables these to be assembled into multi-bay structures. (7) These standardized frames have been fabricated from strip steel as so-called hollow-flange sections. With reference to this prefabricated steel structure I should like to explain that mass production involves more than the mere standardization of types and of components The primary aim of the creative engineer is to achieve greater economy with adequate structural safety. In using steel for building construction there are, initially, two ways of arriving at an economic optimum, namely, by increasing the strength of the construction material and by attaining optimum stiffness conditions as a result of appropriate design. With regard to increasing the strenght of structural steels, the controlled strain-hardening associated with the cold-working of strip steel, in accordance with proposals by Professor Klöppel of the Technological University of Darmstadt, has acquired importance in recent years. It had, long been known that the strength of wires and round steel bars could be substantially increased by cold-stretching or cold-twisting. These methods have been used for many years in the manufacture of suspension bridge cables and of tendons for prestressed concrete. A new feature, however, was to apply strain-hardening to structural sections made of strip steel and to do this in such a way that, simultaneously with the forming of the cross-sectional shape, the sections are uniformly strain-hardened over almost their entire area or, alternatively, that controlled strain-hardening is applied to particular cross-sectional parts where it is desired. This strain-hardening process by means of cold-working is, like many other physical and chemical reactions, time- and temperature-dependent and is associated with a simultaneous change in other properties of the steel. Just as, for example, in medicine, an antibiotic taken in excess will act as a poison, it will, when taken in the correct dose act in certain parts of the body and have a beneficial effect, so strain-hardening, too, must be applied only with methodical planning. Also with cold-working it is possible to adopt cross-sectional shapes which, with regard to structural design, attain the optimum rigidity to the purpose for which they are intended. This modification of the cold-forming technique of steel fabrication can therefore appropriately be designated as "strain-hardening *with* optimum rigidity".

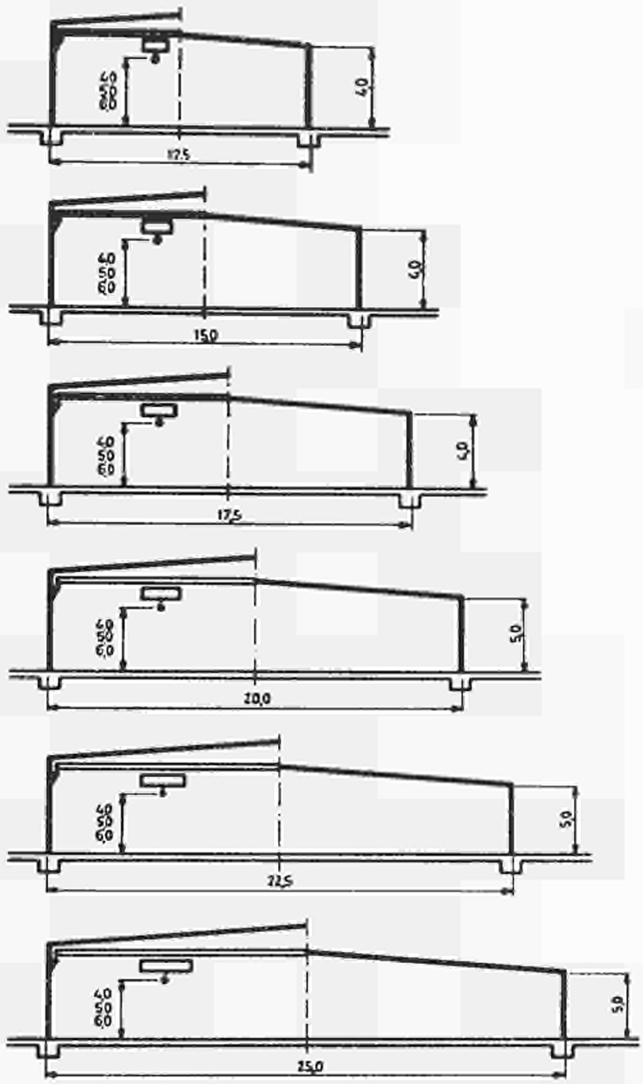


Fig. 5

But what do we understand by strain-hardening (strengthening by cold-working) in the metallurgical sense? Of course, in the solid state steel is a crystalline material, *i.e.*, its atoms are arranged in a particular space lattice. The atomic ordered state is never ideal, however, but it is instead — because of the rate of solidification of the molten metal — disturbed to a greater or less extent. As distinct from the regular and symmetrically constructed "ideal crystal" that can be conceived according to the laws of crystal structure, the "real crystal" of a metal exhibits a number of disorders of a chemical, structural or electrical character. The principal type of fault to account for strain-hardening is the "dislocation". In a quantitative consideration of the theoretical shear strength of an imaginary "ideal crystal", calculations based on the atomic bond energy yield values which are of an order of magnitude of 10^3 higher than the known measured shear strengths of "real crystals" encountered in actual practice. The explanation of this apparent anomaly must be sought more particularly in the existence of atomic disordered states, especially the type of fault known as dislocation.

The dislocation theory which has been elaborated in the last two decades, explains the strain-hardening of metal as a progressively increasing restraint — developing with increasing deformation — of the dislocation movement. The dislocations initiated when the yield point is reached will, in their movement, encounter other dislocations, these being already present as a result of crystal growth and having more particularly been formed in ever increasing numbers by the deformation, so that they pile up, as it were, and obstruct further slip. Carbon atoms, which collect chiefly at points of disturbance, and especially at dislocations and grain boundaries, as so-called carbon clouds, additionally anchor the "piled-up" dislocations and prevent them from undergoing further movement (which would correspond to plastification of the steel) until a higher stress is applied.

Since the strain-hardening of single crystals is also dependent upon the orientation of the direction of stress, the transition to the polycrystalline metal represents — apart from the effects arising from foreign atoms — a highly complex average effect of the influences of the statistically random oriented individual bodies with their inhomogeneous states of stress.

Practical application of the theory of strain-hardening by cold-working has been made in the case of nearly all the prefabricated steel components referred to in this paper. In the standard structural frames for shed-type buildings both the cold-formed hollow flanges and the strip-steel web (joined to the flanges by means of automatic submerged arc welding) are subjected to controlled strain-hardening. With St 37 as the initial material, yield point values are attained which correspond to those of steel grade St 52.

The modification of the cold-working technique for strip steel, as described here, is called "strain-hardening *with* optimum rigidity". In the case of relatively thin-walled structural members loaded in compression and bending, there is little point in increasing the strength without devoting the necessary attention to the stiffness of the members so as to obviate the danger of instability. For this reason the hollow-flange girder combines the highest load-bearing capacity of the I-section for loads acting *in* the plane of the girder with the adequately torsionally rigid construction of the hollow flange for resisting loads acting at right angles to the plane of the girder. It is of interest to carry out an analysis for lateral elastic instability in the case of a two-pinned portal frame for a shed-type building. The top flange of this frame is laterally restrained by purlins, but the bottom flange (which is loaded in compression at the corner of the frame) is not restrained. This corresponds to the arrangement usually encountered in practice. Using the energy method to solve this stability problem, with the energy indifference criterion:

$$\delta\,(\delta^2\pi) = \delta\,D\,d\,z = 0$$

we obtain the criterion for the safety against lateral instability γ_k for an immovable axis of rotation. As a practical example we can consider the theoretical dimensions of a portal frame of 82 ft. (25 m) span, subjected to a uniformly distributed load comprising the dead weight of the roof and snow load. On comparing a hollow-flange section and a welded I-section having the same cross-sectional area and approximately the same moments of inertia, we obtain for the hollow-flange section, after due allowance for plastic lateral instability, exactly the factor of safety which is considered to be adequate by the German Standard Specification DIN 4114, namely, $V_k = 1.71$. For the single I-section the corresponding value is $V_k = 0.34$ and therefore inadequate.

Manufacture of prefabricated steel components on automated production lines

At the present time, in the stage of transition from building by artisan's methods to industrialized prefabricated production, it is no longer permissible to judge the structural components merely by themselves; instead, they must be considered in close relationship with the method of manufacture. It would appear to me that on the way towards rationalisation of construction technique it is, irrespective of the different properties and effects of the various materials, necessary that the following essential, though not entirely adequate, requirements be fulfilled:

(1) Standardization of large structural components within a manufacturing programme.
(2) Adequate fitting accuracy and architecturally satisfactory concealment of the joints at the edges of the components.
(3) Interchangeability of the components.
(4) Inclusion of the additional properties — weatherproofing, thermal insulation and acoustic insulation — in the structural components.
(5) Mass production of the components on automated production lines.
(6) Mechanical and electrical interlinkage of the individual stages of manufacture.
(7) Programmed control of manufacture within the programme of standardized types.

If we consider industrialized building production in this sense, we find that, among the most frequently employed construction materials — stone, ceramic products, timber, plastics and metals —, steel is best suited for production-line manufacture, because of its outstandingly good properties — particularly its rapid cold-formability. The only factor that acts in opposition to mass-production of prefabricated steel components by such methods of manufacture is the relatively high capital outlay involved in establishing such production lines. The financing of technical progress, however, is after all not a special worry affecting the steel fabricating industry alone, but is a problem currently affecting the entire European economy. The prefabricated steel components have recently gone into production on six automated production lines in a new factory at Hamm.

I want to conclude this lecture by citing the words of a great architect:-

> "A great age has dawned,
>
> A new spirit is abroad in the world.
> Industry, turbulent as a river striving towards its destination, brings us the new resources which befit an era filled with a new spirit.
>
> Heavy industry will have to concern itself with building construction and mass-produce the individual components. Mass production is based on analysis and experimental research.
>
> It is essential to create the right state of mind to enable mass production to be achieved.
>
> When we dispel from our minds and hearts the rigid conceptions of a house that have become established there and consider the matter critically and objectively, we are bound to arrive at the conception of the house as a tool, the standard house, which is healthy (also morally healthy) and just as beautiful as the working tools that accompany us in our existence.
>
> Also, beautiful, thanks to the animation that artistic feeling can give to austere and purely functional tools".

I am sure you will agree that these words are most appropriate to the present time. Yet they were written more than forty years ago by no less a man than Le Corbusier, who thus clearly stated his position with regard to the so often adduced argument that standardisation of types and mass production in building construction must lead to monotony. Le Corbusier declares:- "Mass production calls for uniformity of the elements and permits animated variety of the whole"; he continues:- "Exactly the opposite of what we are now achieving: mad diversity in the elements and dreary monotony in the streets and cities."

Description of photographs

1 — Steel roof which functions as an orthogonally stiffened plate-type structure.

2 — Strip-steel floor.

3 — Various prefabricated wall components.

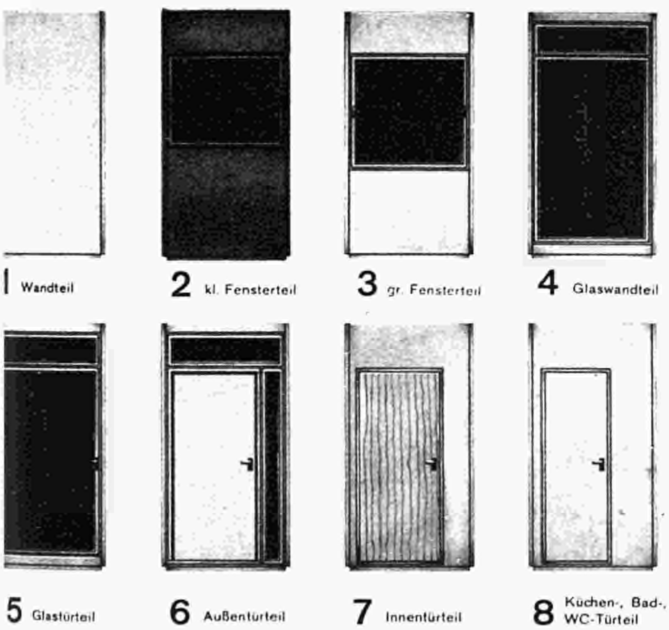
4 — Types of detached single-family houses.

5 — Administrative building (hall) — Example of plastic-lined steel prefabricated components.

6 — Façade components strenghthened by means of rectangular tubular sections.

7 — Shed-type buildings — Multi-bay structures.

2



1 Wandteil
2 kl. Fensterteil
3 gr. Fensterteil
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5 Glastürteil
6 Außentürteil
7 Innentürteil
8 Küchen-, Bad-, WC-Türteil

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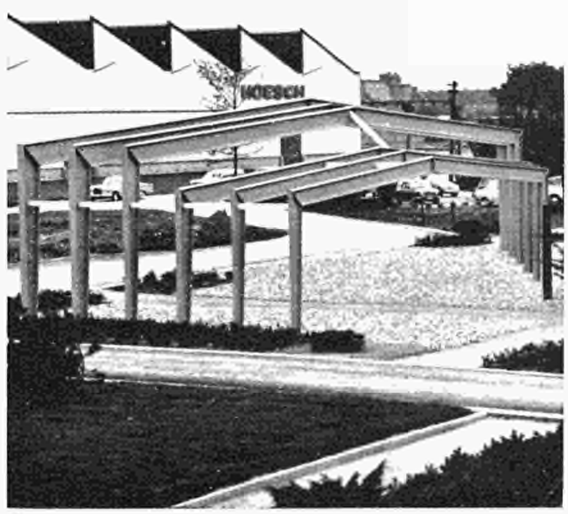

7

Jacques LORMAND

Industrialization of Housing Construction Large-scale Production

(Original text: French)

As Monsieur Wahl observed when discussing prefabrication, in his very detailed paper, truly the industrial methods of production can be considered in the building industry. For this industrialisation, steel can play a large role in the construction market. For most people, industrialization implies mass production, which is looked upon as the first condition of all industrial development. It is necessary to consider some of the various aspects of this industrialization, notably the need,

(a) to refer to the demand

(b) to give thought to the price structure

(c) to have a commercial organization.

With regard to *demand*, contrary to that which takes place in the field of durable consumer goods, the need for a dwelling is not directly tied to the fabrication of building components. The demand is transformed, modified, interpreted at the various intermediate stages (promoters, architects, design offices contractors, local authorities, etc.) Where is the real demand created in this chain? I feel that the architect is the essential link.

As for the *price structure*, one can observe that the efforts made in prefabrication in recent years do not take into account all the aspects of industrialization (fabrication, storage, transport, erection). The initial cost of constructional elements, steel in particular, should be studied in a much wider fashion, that is to say taking account of all these aspects.

Finally, the third point is *commercial organization*. The need to have an organized commercial service, the obligation to provide service after sale and to set up a guarantee system is already commonplace in industry, but it must also be borne in mind when one deals with the building trade.

It is interesting to note the various ways in which industry can impose itself on an artisan market. One can draw an analogy with the pharmaceutical industry which has passed from its former artisan state and today has attained a remarkable degree of industrialization. In effect, the "artisan" chemist, like the contractor, is governed by traditional working methods. Then there is the doctor, whose liberal profession is comparable to that of the architect, who prescribes and initiates the act of buying. Finally, there is the consumer who, like the tenant, has little influence on the presentation of the product. This picture is interesting in that it enables one of the points of decision to be determined. But it is also interesting to pursue the analogy in order to follow the history of this evolution from craftsman to industrialization. In this respect, three facts are worthy of note:

(1) The characteristic of this industry is its work in close collaboration wich the doctor whith is similar to the relationship between fabricators of steel products or prefabricated members and the architect, that is to say, *a commercial function via scientific advice.*

(2) The presentation of a *product which is more highly developed than that which the craftsman can produce*, necessitating the study of members which are prefabricated to a high standard.

(3) *Research which is designed to sustain simultaneously commercial profitability and the development of such new products.*

"These very simple ideas should be kept always in mind. They imply certain efforts to resolve the three essential problems involved. The first is "getting the market started". It seems that only the dynamism of several firms, through a much greater degree of competition, could unleash a development of this nature. Similarly, numerous complementary sectors of industry *e.g.* steelwork and the plastic chemical industries, could join in association in this movement, having common interests which could be turned to good account.

The second point is geographical freedom. In the field of heavy prefabrication, it is evident that an industrial undertaking — *i.e.* one engaged in mass-production — cannot find sufficient outlets in a highly localized market. A more highly developed industry must require a much larger market. Swiss industry, for example, produces products for the whole world. In the sector which we occupy, industrialisation calls for a more sustained effort in modular co-ordination. The producers, being the first people concerned, should direct all their energy in this direction.

Finally, technical research must promote the idea of "design-construction", which is at present only in its infancy. It is, nevertheless, interesting to note that architects themselves feel that this is the direction in which they should progress.

Johannes Franciscus HEIJLIGERS

Full Industrialization and Prefabrication

(Original text: Dutch)

It is important, in this connection, to distinguish between the fully industrialized construction of dwellings and perhaps of factory sheds, and the prefabrication of building components for smaller projects, perhaps not even on a mass-production basis if the series form part of the building.

Here and there efforts are being made to confine prefabrication to motor-car and aircraft production. I do not believe in this. I do not believe, either, in the idea of completely ready-made houses arriving at the site by lorry, and I do not think the danger of this is all that great. I feel it would be extremely tiresome if one were to regard dwellings as mass-production commodities.

We must try, as architects, to think more in terms of prefabrication. This means that knowledge of the material, which used to be limited to familiarity with the actual raw materials to be employed, must now be extended to include knowledge of the product as it leaves the factory; of how that product is manufactured and how long its manufacture takes, so that the modern architect needs to extend his field of knowledge quite considerably.

I think the architect should remain the leader of the building team. His requirements as to form will affect the costs. There is a certain interaction among design requirements, cost requirements, production methods, assembly time, assembly possibilities and design. For these to run smoothly there needs to be co-operation from an earlier stage than has up to now been customary. I would urge the closest co-operation right from the beginning of the preparations between the architect, the builder, the contractor and the manufacturer; I hope, also, that something may be done through education and information to make builders more steel-minded than they are at present.

Henri DE LASTOURS

Comments en Standarized Portal Frames

(Original text: French)

I should like to comment a few photographs of standardized portal frames for industrial sheds.

A team of some ten French designers have evolved a reduced series of portal frames in which the aim is to get the maximum performance out of the comparatively new IPE section. These frames all consist of IPE columns and rafters, the extensions being of steel sheet welded in the areas of highest bending moment. The standard widths are 12, 15, 20 and 25 metres, and the heights 4, 6, 8 or, in exceptional cases, 10 metres.

As you will see in photo 1, the sheds can be paired together. They can also take craneways for travelling cranes; the ones shown here have craneways, the consoles for which can be seen in the foreground.

Photo 2 shows a number of assembled sheds. As you see, they can be fully finished, clad with a great variety of materials, such as asbestos cement, or aluminium, or galvanized steel sheet — in fact by all sorts of processes.

Photo 3 shows two sheds coupled side by side and a third seen end on.

Photo 4 shows a shed with a travelling crane already in position and ready for operation.

Photo 5 is included mainly to give an idea of the lightness and cleanliness of outline offered by these standardized portal frames.

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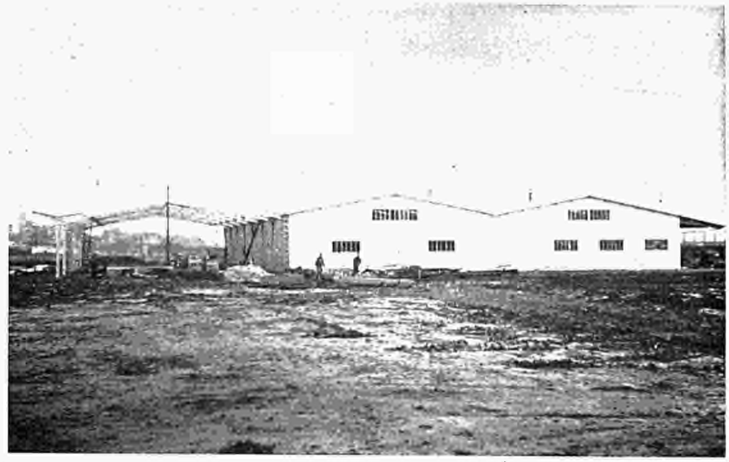

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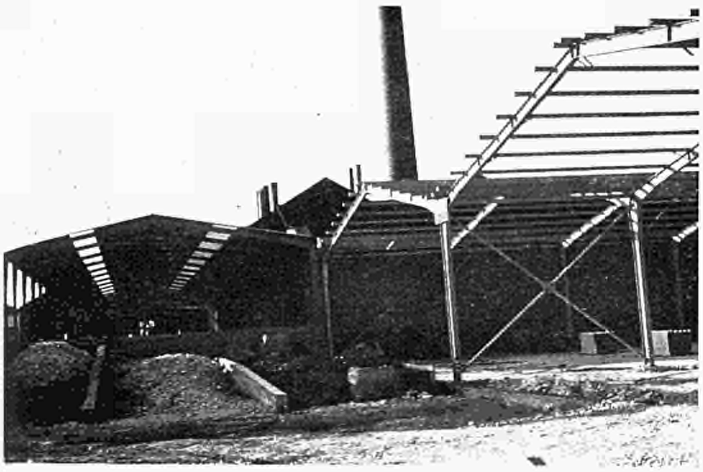

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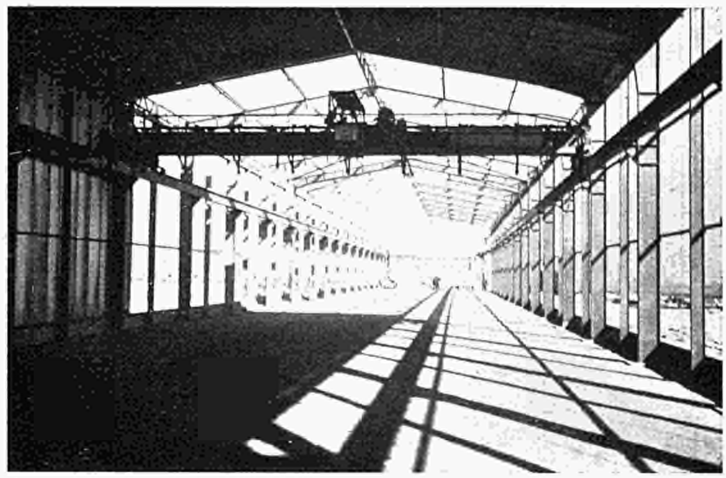

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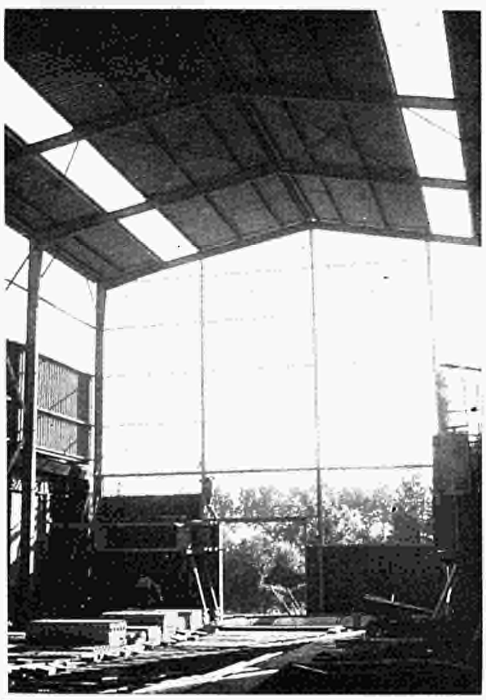

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J. HEINEN

Studies and Industrialization

(Original text: French)

As I see it, industrialization is not just a question of extending mass production and standardizing components or simplifying them as far as possible: it is above all a question of designing them so as to establish production series. To do a proper job on a programme for the industrialization of building, there would need to be a team consisting of architects, engineers, time and method study experts, and production engineers. One big question is, who is going to pay for all these studies? The technical possibilities have been extensively gone into and referred back for further study a dozen times. No spectacular breakthrough is really to be expected now. As regards *préfabrication ouverte*, I think only intuitive inspiration could provide a short-term solution.

In the matters we are dealing with, study is the great thing, and must obviously precede any attempt is made to industrialize. In an independent designer's office, the question is constantly arising, who is prepared to take over the substantial load represented by the preliminary studies? So far, the only people who are in a position to see them through are groups of industrialists, and all groups of industrialists are naturally bound to have a speculative approach to industrialization. It should be the job of us consultants to see that the value of such techniques is brought out in their economic and qualitative respects, for the general benefit.

I do not think there is any immediate prospect of bringing costs down appreciably, until markets are larger and industry is assured of orders spreading over a period of years.

Incidentally, I have noticed in certain specific cases that this industrialization, or at any rate standardization, was not altogether in line with the size and module standards recommended by the European Organisations. However, in view of the very large capital sums that are being laid out, and the important results being obtained, I do feel these groups really will achieve industrialization, even if the end result does not entirely measure up to the desired ideal.

Henri WAISBLAT

The Part played by Stainless Steel in Prefabrication

(Original text: French)

Post-war European architecture has fostered developments in the use of thin steel sheet which have opened the way to the employment of stainless steel sheet. It should be noted from the start that stainless steel sheet is not in competition with black, galvanised or vitreous enamelled sheet; on the contrary it is a complementary material which widens the field of application of steel to a great extent. This is particularly true for cladding where steel completes or replaces masonry more and more.

Stainless steel in the form of mullions, weatherproofing and an assembly of panels and fitting serves as exterior cladding or a skin or a wall in which the structure is in steel. The tonnage of steel at present consumed for example, in certain types of curtain walls, where only stainless steel is visible, often exceeds the portion of stainless steel concealed inside structures. Stainless steels are therefore steels heavily alloyed with chromium or chromium-nickel, which are finding constantly increasing applications, thanks to the judicious and rational conditions under which they are employed, either in the form of sheets or coils or as bars, wire and flats. Despite the fact that stainless steels were formerly looked upon as luxury goods out of the range of normal structures, they are now embodied into every type of structure, in roofing or in cladding, whether for offices, schools, hospitals and low-cost housing or even for industrial buildings. The evolution which has widened its scope has resulted from research made with the following aims in view:—

(1) To choose the quality of material and the surface finish most economical for the service required. Only a better knowledge of the working and ageing conditions can help us. For example, all other things being equal, it is more sensible to select an adequate surface finish produced under good technical conditions and to allow for periodical cleaning, than to search for more highly alloyed, but more expensive, steels.

(2) To use the thinnest sheet compatible with the geometrical characteristics of the members and the manufacturing conditions concerned. The progress made in this field now allows us to use gauges as fine as four or even three-tenths of a millimetre. The methods of use of thin gauges, although derived from those for normal steels, have been brought to a fine art for stainless steels, to take account of their mechanical properties, their appearence and the regulations for surface protection. As a general rule however, it may be said that stainless steel sheets, cold rolled, can be fabricated in accordance with practically all the sheet metal techniques. At the present time in France a certain number of processes have been brought to a pitch unrivalled qualitatively or quantitatively elsewhere in Europe. These processes are concerned not only with traditional techniques, but especially with industrialized systems. First, we will consider simple products, the high quality of which is obtained with little workmanship, such as sheeting and cladding, self-supporting long-span roofing sheets, all kinds of profiles formed in presses and then products requiring much more labour, such as casement windows and the framing and panels for curtain walls. The photos illustrate some recent examples.

In conclusion, and to establish the magnitude of the use of stainless steel in French architecture, it should be mentioned that this market opened only a few years ago. In 1963, the consumption of steel was about a thousand tons but already in France there is nearly 2 million sq. ft. of stainless steel roofing and about a million sq. ft. of light cladding or fittings in stainless steel. These results have been obtained with the assistance of the roofing contractors who, although they are generally conservative craftsmen by repute, have shown that should the need arise, they can employ new materials with excellent effect. By contrast, in the case of cladding the number of firms represented is comparatively small. It is to be hoped that with greater experience the number will increase, because stainless steels can provide quality in the industrialisation of building.

Description of photographs

1 — Facade of the new terminal building at Orley airport, near Paris, consisting of stainless steel curtain walling and comprising opening windows and a framework the mullions and transomes of which are faced with 18-8 stainless steel.

2 — Details of the facade of the Orley building. Only the stainless steel sections are visible above the glazing.

3 — Two large structures in the Paris area shown together: the Centre National des Industries et des Techniques (CNIT), in which stainless steel members have been used for the large glazed area, and an administrative building, the whole of the curtain walling for which is in stainless steel.

4 — A further view of the administrative building which comprises nearly 80 tons of stainless steel curtain walling. The frames are in hollow sections, fabricated in a press and mechanically assembled without welding.

5 — The different erection operations for the curtain walling in this office building may be seen from left to right. The facade is almost complete on the right. On the left only the framing may be seen. Then the curtain walling is being erected with cill panels in cellular galvanised steel sheet and with stainless steel windows.

6 — Facade of the Centre National des Industries et des Techniques: details of the stainless steel members. This is a section in medium tensile steel with very high mechanical properties, the tensile strength being 63-66 tons per sq. in. Working conditions have made it possible to stress this material up to about 20 tons per sq. in. in mullions made in three sections each 1.25 mm. thick, comprising secondary framework supporting the glazing.

7 — Blocks of apartments in which the cill bands have a stainless steel skin. The panels are fabricated by a mass production method in which polyurethene foam is injected into cellular frames, the outside wall of which consists of ribbed stainless steel sheeting 0.4 mm. thick.

8 — Details of this facade. The panels made in this way are very light, weighing less than 5 lb. per sq. ft.

9 — A more original method of cladding the exterior. Very simply, it consists of pressed steel sheet, 1 mm. thick, for an office building.

10 — Industrial building with an original type of cladding. The light panels are bands of stainless steel, pressed sheets 0.4 mm. thick and 10 m. long; the dark areas comprise translucent polyester panels.

11 — Another industrial building with bands of the same profile, but without a transverse joint. This is a method using chromium-nickel steel.

12 — Stainless steel roofing with vertical joints, a system much favoured in France. A certain number of roofing contractors nowadays use long bands of stainless steel with a chromium content of 17%, the technique being the same as for other metals.

13 — Office building in which all the cill bands are pressed stainless steel troughing 0.6 mm. thick.

14 — The same building as in Photo No. 13. The panels, which were fitted into a steel framework, were completely fabricated in the shops, protected by a vinyl coating, despatched and erected on the site, after which the coating was peeled off to leave a completely undamaged surface.

15 — Details of this facade, in which the stainless steel cills and window frames and a whole range of sections may be seen.

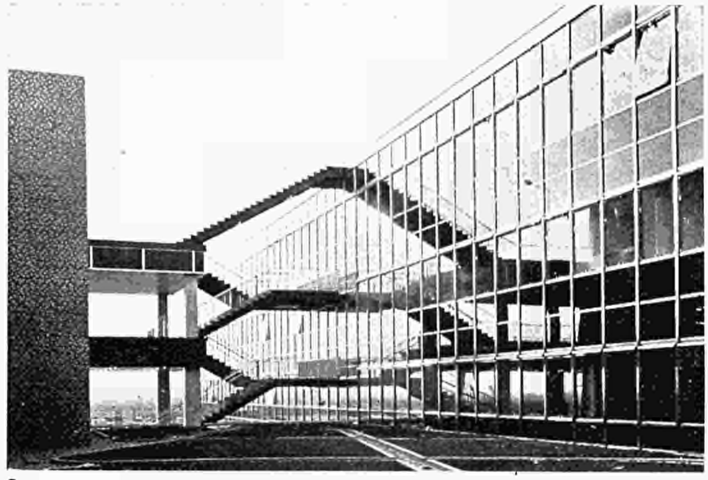
16 — View of facade of offices showing stainless steel mullions and transomes.

17 — University building At Strasbourg with stainless steel cills.

18 — Faculty of Medicine in Strasbourg.

19 — Factory in Seine-et-Oise. Stainless steel sheets and fittings in 18-8 stainless steel.

20 — Details of building in no. 19.



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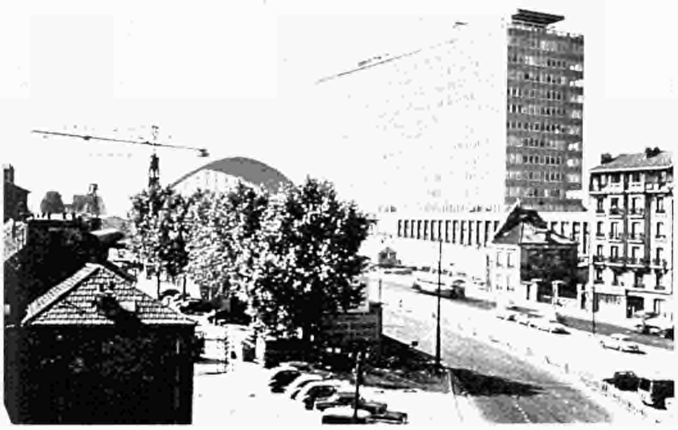

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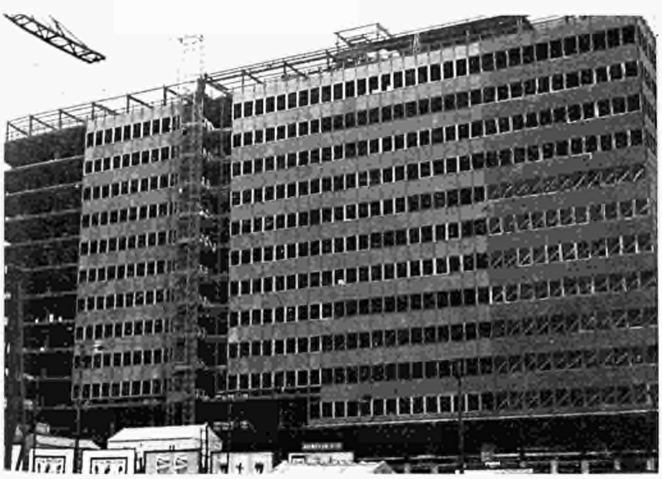

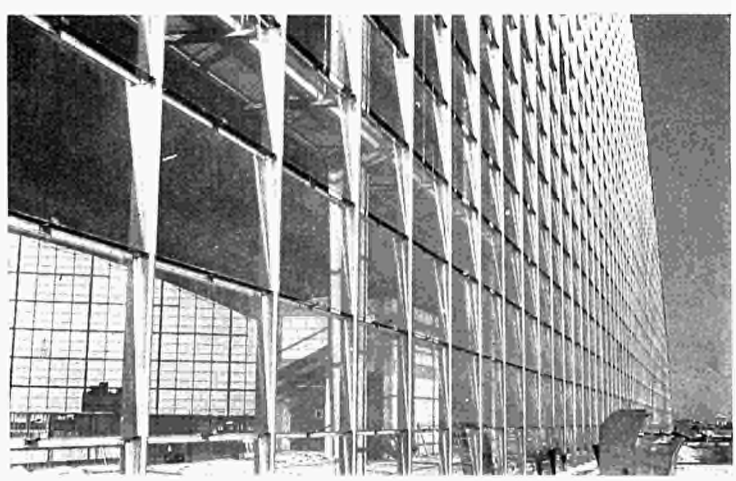

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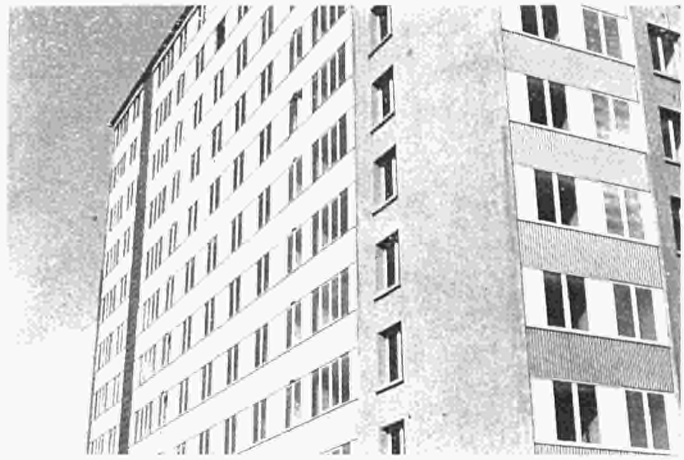
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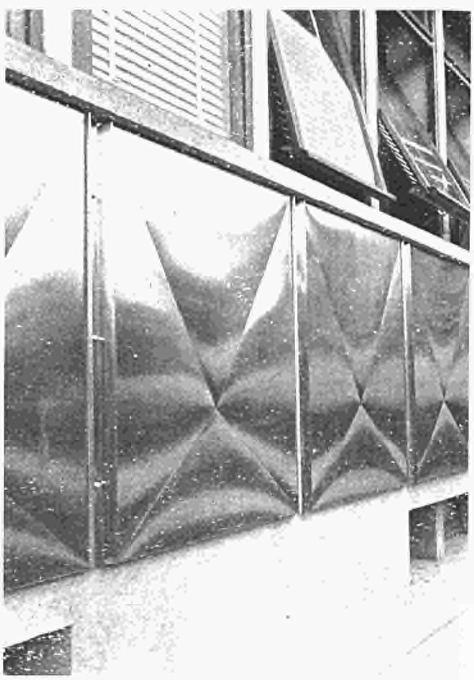
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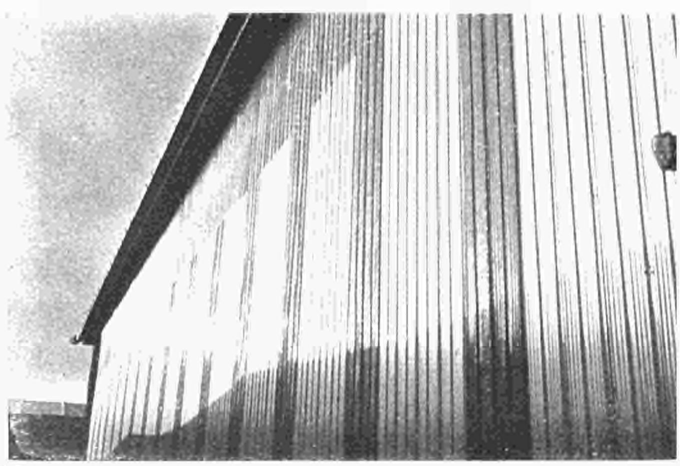
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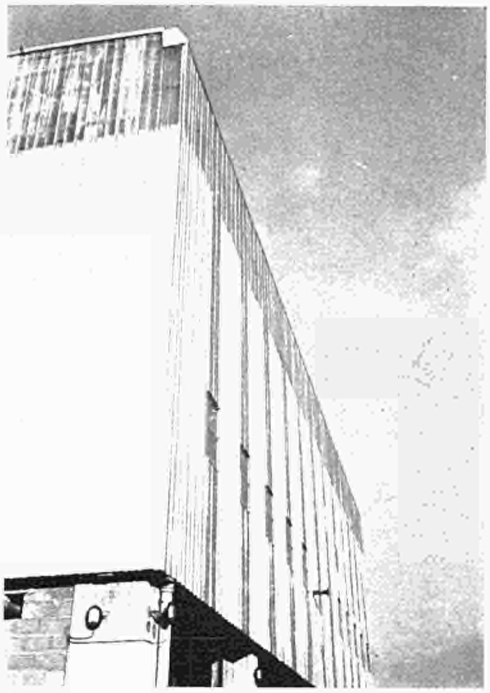
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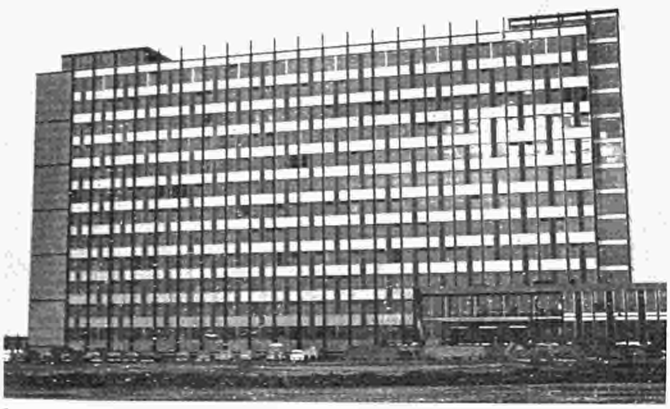
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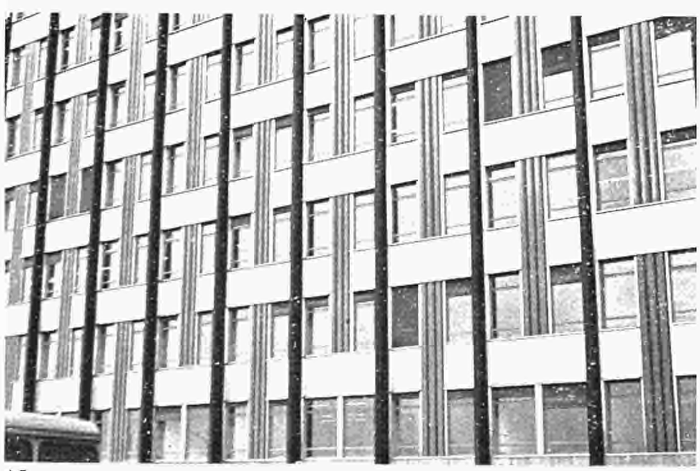
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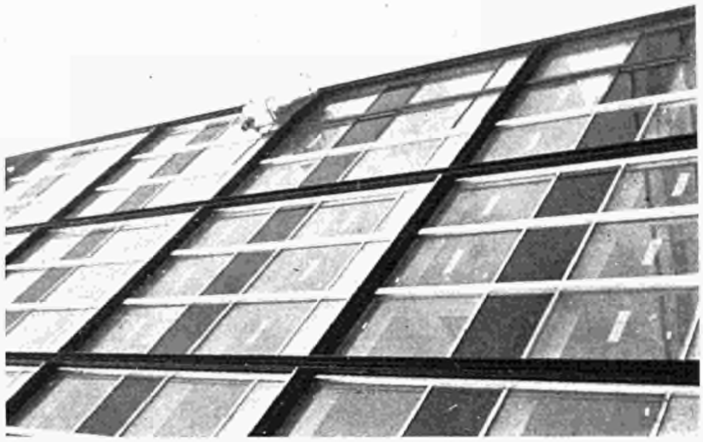

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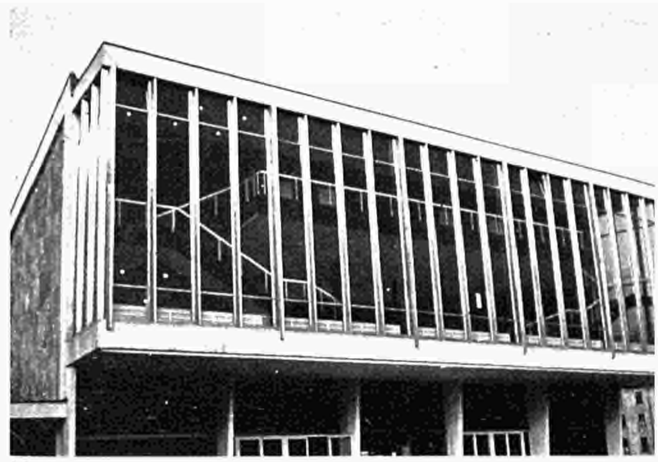

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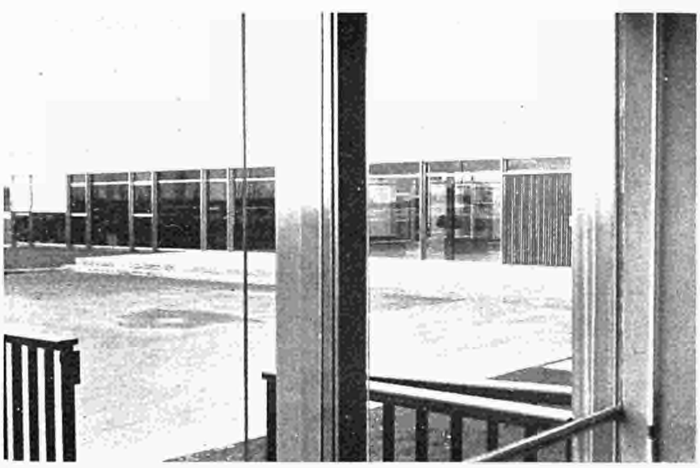

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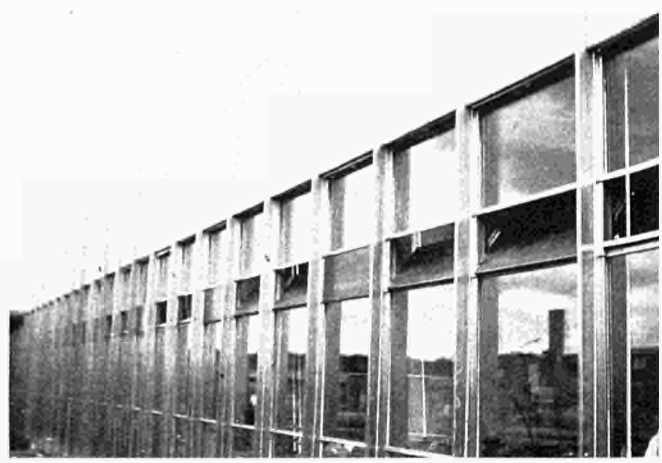

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André NOÉ

Factors affecting the Resistance of Stainless-Steels to Atmospheric Corrosion

(Original text: French)

The use of stainless steels in building must be considered from several aspects, which are not necessarily independent of each other: mechanical behaviour, aesthetics, and resistance to corrosion.

This document will confine itself to the last of these aspects. Furthermore, it will only touch lightly on the subject and will be limited to expressing the conclusions at which we have arrived after several years study of the problem, the subject being more widely covered in a publication to appear in the near future. [1]

General

The main characteristic in the behaviour of stainless steels in the presence of natural, *i.e.* respirable, atmospheres, is that their corrosion is not facilitated. In more technical terms, one can say that the critical conditions occur at an elevated potential which is not attained in the majority of cases. Where this figure *is* reached, the corrosion currents are so weak, that one can only confirm the presence of some pitting [2] and, sometimes, of fine, non-adherent deposits: the eventual corrosion of stainless steels is only a surface phenomenon and takes place without any measurable destruction of the material itself. However, the study of it is of great interest, as its mechanism assumes a quite singular character. We shall consider in turn the effects of the surroundings, the type of steel and the treatment of the surface.

The surroundings

The part played by the atmosphere has been fully expounded by numerous authors in relation to current steels in normal usage, but much less so in connection with stainless steels. [3] [4]

The general opinion is to hold the sulphur in combustible materials responsible for the majority of corrosion phenomena. Our view, on this point, is that the sulphuric anhydride present in the drops of moisture is too weak to have a direct action on stainless steels.

On the contrary, we place a greater stress on the presence of chlorine and chlorine compounds in industrial and urban atmospheres as much as pollution by solid particles: soot, metallic oxides, carbonates, silicates all these elements are avenues of corrosion by pitting, although the process may vary widely from one to another.

Type of steel

It is well known that nickel slows down the reaction, assistinб considerably the resistance to corrosion by surroundings as variable and changeable as the atmosphere. A steel of the 18-8 type is to be preferred whenever visible surfaces are concerned.

As for molybdenum, it reduces without doubt the possibility of pitting for a given surface, but the better the treatment which the surface has received, the smaller is the influence of the molybdenum, because a proper treatment of the metal is also efficacious in reducing the possibility of pitting.

Surface Conditions

We accept freely that the care with which the surface of stainless steels to be exposed to the atmosphere is treated, governs to a considerable extent the behaviour of those steels. This is a fact derived from experience. We have been able to confirm that a sample of 17½ Cr. Steel (type 340), situated on the sea coast, and which had received an electrolytic polishing, behaved as well as, if not better than, a sample of 18-8-Mo steel (type 316) with a 2B finish.

Theory allows one to foresee fact. Atmospheric corrosion takes place by means of droplets of condensation, as shown by Professor Evans. [1] It is exomorphic, *i.e.* brought about by discontinuities in the surroundings, and therefore not idiomorphic. That is why it is necessary to take care not to transpose to atmospheric corrosion the results, just as they stand, which have been acquired in the study of idiomorphic corrosion, where the surrounding corrosive medium continuously washes the metal.

Thus, the eventual corrosion is due to the presence of droplets of water containing dissolved products which form an electrolytic conductor. Further, these droplets can contain impurities capable of incuding pitting. Finally, a certain time must elapse for the passive state to disappear locally and the active metal to begin corroding.

Consequently, great importance is attached to making the metal as passive as possible, followed by giving it a surface geometry as fine as possible, in order to increase the resistance to pitting and to promote the physical elimination of water droplets before their effect can make itself felt.

Conclusion

Construction in stainless steel has its rules, just as have other forms of construction. It must be noted that too much care cannot be paid to the preparation of the surface (making passive, cleanliness, smoothness) and to its maintenance.

The architect, for his part, must take care to eliminate all useless protrusions in designing his frontages, and would be well advised to include in his specification suitable protective measures against incidents in the yards and a general cleaning once work has been entirely completed.

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Giorgio RIVA

Prefabrication of Steel Building Components

(Original text: Italian)

Although the subjects dealt with in this paper mainly concern stainless-steel, one or two cardinal aspects of the general problems involved in the prefabrication of building components are also discussed.

As a person engaged in Italy both as a practising architect and a research worker, I have two main lines of argument to put forward, the first concerning technological training in the use of prefabricated components and the second relating to humanist education, particularly the maintenance of a progressive architectural outlook, in the designing of mass-produced components.

Introduction

Nowadays, even in Europe, building design relies on approved mass-produced components such as beams and floors and, as regards stainless steel, door and window frames fittings, cladding, and panels, etc. These components are the outcome of prior design work concerned with the processing of the raw material (billet or strip) into a mass-produced item often possessing inherent architectural properties, *e.g.*, a rolled profile. There are thus at least two design stages, one for the component and one for the building project as a whole.

We must therefore consider the question of linking these two stages as part of a single objective, namely the large-scale production of building components whose value depends on their ability not only to meet technical and economic requirements but also to respond to the social, psychological and environmental needs which must be satisfied if an object is to be culturally progressive if not specifically innovatory.

Arguments

On this basis, the following questions offer scope for discussion:

(1) In the stainless-steel sector (and elsewhere) it often happens that a product intended for building purposes does not meet with the approval of the building designer. This may be due to two reasons:

(a) The designer is unfamiliar with the material (stainless-steel) and ignorant of its characteristics, *i.e.*, his level of technological knowledge has not kept pace with industrial discoveries;

(b) The prefabricated product being incapable of meeting the problems to which modern design demands a solution, is therefore not considered suitable for use in a building, however brilliant an answer it may represent technologically.

(2) How can building designers keep abreast of technological developments? Is the present huge volume of advertising material sufficient for this purpose, seeing that it represents special interests and is therefore an unsuitable means of studying problems of method? Are the few specialized publications available, sufficient for the designer who is facing new problems often involving many new difficulties?

If existing means of disseminating technological information are inadequate, then specialized institutions and media will be needed for this type of advisory work.

(3) Let us now consider the way in which a prefabricated component, designed purely to solve technological problems such as strength and corrosion resistance, can perform other functions when called upon to serve in a structure built for human needs.

Where it is of secondary importance (a substitute and replaceable, *e.g.*, decorative mouldings) there is no problem, but if, as in the case of highly-developed prefabrication, it is a characterising element such as wall panels or door- and window-frames and fittings, the building designer must aim at "humanizing" its technical characteristics by endeavouring within a limited period of time to increase confidence in the use of the material, only achieved in earlier times by prolonged use and contact. This entails research often beyond the scope and means of an architect's office. Also, the component is often so rigidly characterized morphologically as to render useless any attempt at adapting it to human needs.

Here again specialized rather than commercial institutions and media are needed to provide a reliable bridge between the designing of the structure as a whole and that of its components, if the risk of producing superb but partially unusable equipment is to be avoided.

(4) We have spoken of institutions and media capable of ensuring this link between the two design phases, which in our opinion represent two limited approaches to the problem which, in current practice, are constantly in contact with each other on commercial questions but never on general problems of method.

In our opinion such specialized institutions, or research centres, should not merely act as information offices whose function is merely the passive one of transmitting information, but should embody at least the following three characteristics:

(a) a guarantee that the general interest will be represented, *i.e.*, the entire stainless-steel industry and not merely one group of companies;

(b) capacity to stimulate research by the building designers, into the best applications of prefabricated components (technical advice);

(c) capacity to stimulate research by manufacturers of prefabricated components so as to ensure their compliance with all necessary requirements from the very beginning.

All this will entail a systematic study of builders' and architects' problems in the light of market requirements, of the scale of residential housing programmes, and of the latest developments at universities and research centres.

In conclusion, I should like to mention that if an effective link between two design levels is to be established, it is pointless employing practising architects (even if qualified) in prefabricated-component organisations because they would suffer from the influence of an inevitable atmosphere of one-sidedness and partiality, if only as a result of their working environment.

In my capacity not only as an architect but also as a research worker in the field of stainless-steel (building) components, I consider that there is a most urgent need for a forum in which to discuss these questions of method fully and with detachment. In which not only manufacturers, designers and experts in prefabricated steel components but also builders, architects and housing and town-planning experts, should take part.

Description of photographs

1 — Example of a special application of stainless-steel strip for roofing purposes. The prefabricated component is reduced to a mere strip. All bending and welding takes place on site, thus allowing considerable freedom in adapting the material to the design requirements of the building, irrespective of its type, shape and size. This simple application is a striking example of how building design requires site organisation and raw material (steel strip) production, to be able to harmonize.

2 — Detail from photograph 1: neatly designed machinery for rapid and automatic bending and welding in situ.

3 — The same principle as illustrated in photograph 1 applied to a roof with a non-uniform curved pitch.

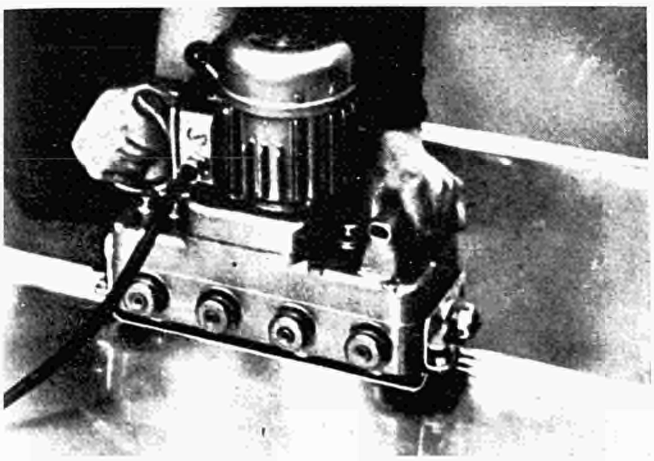
4 — An example of prefabricated stainless-steel roofing components. In this case the manufacturer supplies the finished pressing for fitting and attachment on site. The modulus of the slab and the stipulated roof pitches must be adhered to in designing the building.

5 — An example of prefabricated steel roofing components. Slab and beam pressings are assembled by a typical standardised method which is the same for both types. Solutions of this kind are obviously suitable for buildings which can from the outset be designed with an eye to prefabricated component technique and dimensional requirements.

In cases such as this where prefabricated components determine the character of the building, the problems traditionally solved during the design of the building must also be solved when designing the mass-produced component.

6 — Detail of structure shown in photograph 5.

7 — Detail of structure shown in photograph 5



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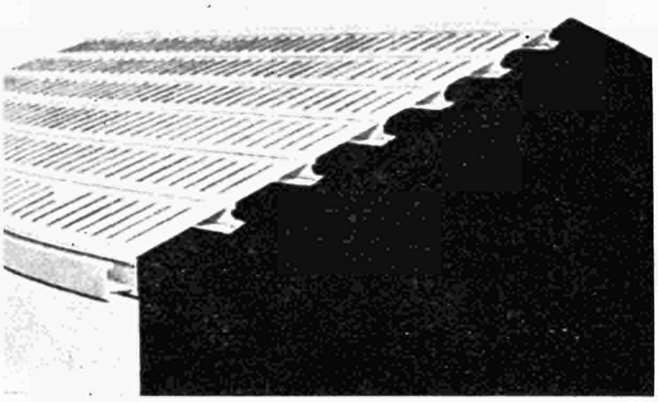

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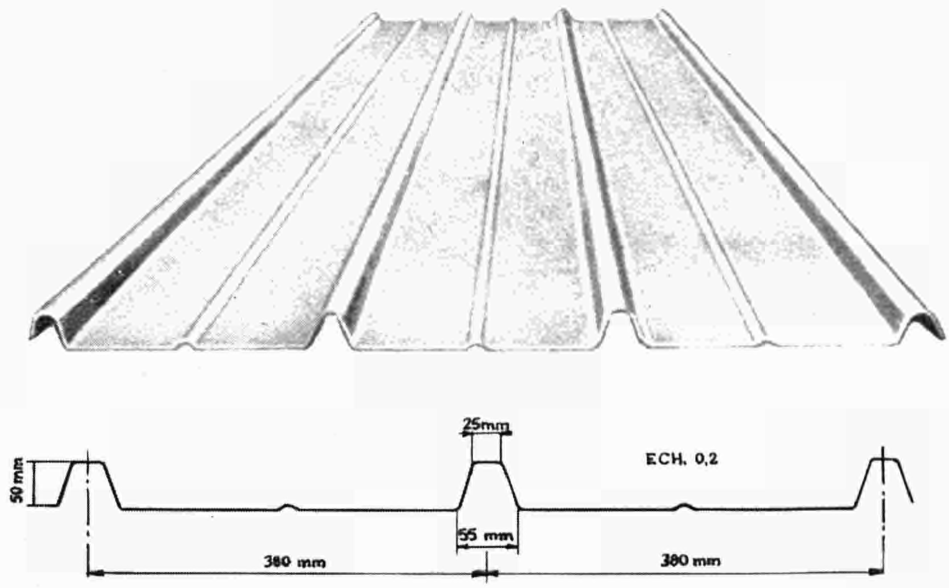

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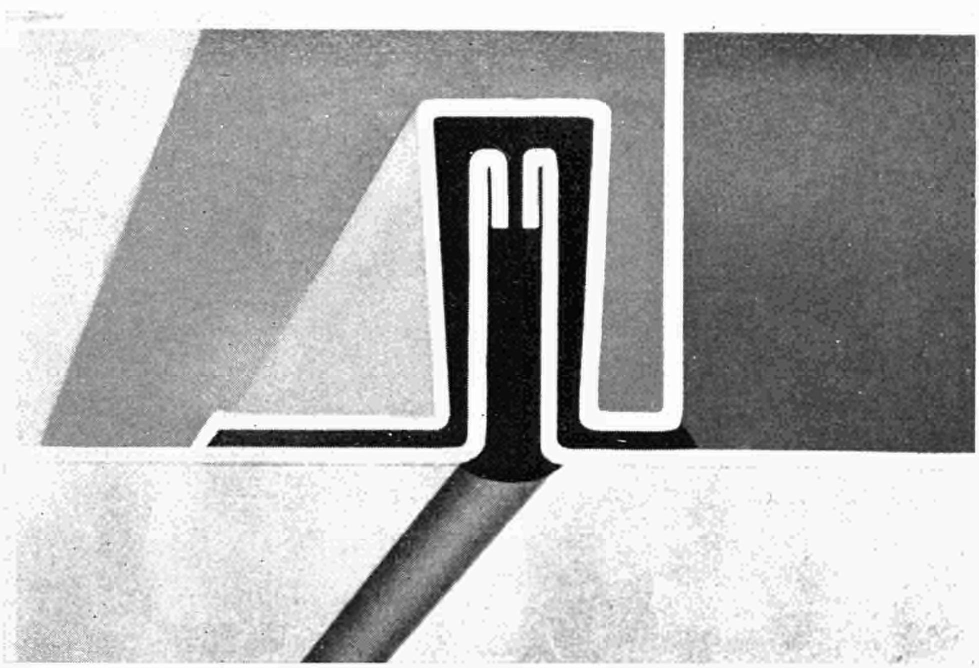
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Roger MORA

Comments on Mr. Wahl's Speech

(Original Language: French)

As a builder, I should like to take up some of the points which Monsieur Wahl discussed in his paper. M. Wahl clearly showed the fundemental difference between open and closed systems. I would add the following:

We builders must study and propose open systems on both the architectural and the technical plane (possible changes of plans and materials).

Perhaps past errors, emphasised elsewhere by M. Wahl, will enable us to discover the prefabrication methods of the future.

In my opinion, these errors fall into two categories:

(1) non-uniformity of fabricating tolerances in assembled materials and weak joints;

(2) ill-considered use of one material in relation to others (*e.g.* thin sheet — steel casing for rolled pillars, subject to corrosion, which destroys its appearance).

Taking these things into account, future prefabrication must consider all possible materials provided that they meet the technical requirements, and steel, instead of being in competition with other materials, will act as a catalyst to their development, giving each one a chance to be used in high-quality, high-precision work.

As for the question of economy, I believe that prefabrication involves two different but complementary economic factors:

(1) economy by mass production, by improving application techniques;

(2) economy by changing working methods, *i.e.* by combining old trades or by creating new integrated teams of craftsmen.

In both cases it is once again precision in the manufacture of the various components which is the decisive factor. The study of the prefabrication methods of the future is, as M. Heinen emphasised an all-trades study by individual trades.

Finally, I come back to M. Wahl's concept of the house, the internal layout of which can be altered by means of movable partitions, and I believe that steel, thanks to all its inherent qualities, is capable of bringing to life the houses we shall build tomorrow.

Gastone GUZZONI

Protection against Corrosion

Original text: French)

I should like to thank Dr. Jungbluth for what he has said about strain-hardening, because if one wishes to make any progress in this field one must return to the classical basis of the theory of deformation, keeping in mind at the same time the physical metallurgy of today.

I should like to add that in a crystalline structure a defective pattern and dislocations alone do not explain strain-hardening, because brittleness at low temperatures, ageing, the transition from the plastic to the brittle state, the presence of several axes, brittleness associated with hydrogen, etc. are also involved.

It would appear, therefore, that metallurgists and engineers should, for the buildings of the future, welding and quality of steels, heat treatment, direct their thoughts to the physical aspect so that they may proceed subsequently beyond the present known limits and exploit todays ideas in the design of the future.

Mention has already been made of the problems of corrosion and the methods, traditional and new, of protection. If I linger over these problems, it is purely to draw the attention of my audience to their importance, because they have a direct influence on the initial cost of steelwork constructions and their maintenance.

If the corrosion phenomena have lost some of their importance, it is only because experience and the putting into practice of more precise test methods have enabled us to avoid in time the appearance of these phenomena in their most glaring forms. However, it would be a mistake to call a halt at the results so far obtained: it is necessary to continue the tests and improve the methods of control and the standards, preliminary or defined, for protection against corrosion. The problem must be studied over its whole extent and manufacturers and consumers must establish a proper collaboration. One must remember that the corrosiveness of industrial atmospheres has increased considerably.

In the study of steelwork construction and in the control of the economic data relating to the cost of construction and the future costs of maintenance, steel could be shown to a disadvantage if the problems of protection against corrosion are not given the importance which they deserve.

On the other hand, corrosion represents the field in which the competitive materials, such as steel, aluminium, plastics, concretes, could join and collaborate efficiently; this collaboration is essential, corrosion phenomena are becoming a catalyst for the alliances and unions of materials and they can have very important developments. The present is but a modest beginning.

The basic protection of plastics or paints, more or less traditional, spray-metallizing processes, the Alu-Process (*i.e.* hot-aluminizing), hot-galvanizing, give varying results from one to the other according to the corrosiveness of the atmosphere, but they also allow a wide choice.

These protections can provide simultaneously acoustic, thermal, or aesthetic effects.

Zinc application remains the most important form of protection, but we are far from exhausting the technical possibilities (for example, the weldability of galvanized structures), whilst from the economic point of view the automatic fabrication techniques of pickling and zinc coating are leading to costs which are becoming more interesting every day.

The actual importance of the problem points to the necessity for a close collaboration between the various corrosion centres, which have already made essential contributions in the field of the treatment of metallic surfaces and, in general, to the campaign against corrosion.

In Italy, there is the U.I.S.A.A. and also the corrosion centre of the Italian Metallurgical Association. But there are also:

— The Belgian Association for the Study and Use of Materials (ABEM)
— The National Council for Metallurgical Research (CNRM)
— The Belgian Centre for Corrosion (CEBELCOR)
— The French Centre for Corrosion (CEFRACOR)
— The O.T.U.A.
— The Netherlands Corrosion Centre
— The German Corrosion Society.
and many others.

There exist studies, organizations, plus experience, but it is necessary to put all this knowledge at the disposal of the constructors, the users, who, sometimes, are small concerns and unable to keep themselves up-to-date with technical progress. There are books and pamphlets which deal with the problems of corrosion and protection, such as:—

— Acier OTUA 1959.
— Pratica delle costruzioni metalliche. Fausto Masi 1955.
— Steel Designer's Manual. Gray, Kent, Mitchell and Godfrey, 1959.
— Light gage cold formed Steel Designer's Manual, American Iron and Steel Institute 1962.

But it is very small in relation to the magnitude of the problem.

The European Federation of Corrosion made an important contribution on the subject last September in its European symposium "Protection of structural steelwork by metallic coatings", and the Austrian Constructional Steelwork Association has published preliminary standards for anti-corrosive protection and new principles on the basis of a review of five categories of protection (Preliminary Guide-Lines for the Calculation and Execution of Light Steel Structures). Dipl.Ing. Krapfenbauer has made a contribution on these protection standards in Vienna and on those (at least of equal importance) concerning the pickling and removal of oxides from metallic surfaces.

Corrosion is a factor which influences thicknesses, safety factors, costs of construction and maintenance. Corrosion is a science and in its theories is found the explanation of all the phenomena which are characteristic of it: the corrosiveness of moisture drops under differential ventilation, the passivity of surfaces by oxygen, and the possibility of damaging these passive layers, the importance of the upper surface, stress-corrosion, intercrystalline corrosion, the phenomena of brittle fracture, of fatigue and corrosion, the phenomena of contact between different metals, etc.

The simple and easy science is at the disposal of everybody and it is only necessary to refer to it to be able to foresee and take appropriate measures.

Cornelis Maarten MAARS

The Problem of Raising the Strength of Steel

(Original text: Dutch)

I should like to offer a few remraks on certain metallurgical phenomena of significance in connection with raising the strength of steel. The cheapest method of raising the strength of steel is, of course, the addition of carbon. This method has the disadvantage, however, that the weldability of the steel is impaired as a result of reduced toughness at or close to the welding points brought about by hardening phenomena. By adding small quantities of a few alloying elements it is possible to increase the strength, and in particular the yield point of the steel without increasing the risk of fracture during welding. From the address given by Dr. Jungbluth it appears that a third method of raising the strength of steel has been developed, based on the strengthening of No. 37 steel by cold deformation so that a yield point like that of No. 52 steel is obtained. While it is true that this cold deformation process results in an increase in the strength of the steel, this increase is accompanied, as the speaker has himself noted, by a loss of toughness. It may even be stated that the percentage loss of toughness is greater than the percentage gain in strength; moreover, it is accompanied by an increase in the stress concentration index and an unfavorable modification of the transition point. I cannot help wondering what remains of the strengthening at points subjected during welding to great heat, which causes recrystallization of the steel and a reduction of the yield point to the original value or even lower. I would also point out the possibility of a critical crystal growth as a consequence of the welding and the increased brittleness at the welding joint that usually results. Penetration near a weld has a very unfavorable effect on the stress concentration index, with the result that spontaneous fracture can occur even with slight reforming, something which I have myself already experienced once or twice when working welded cold-drawn sections.

Another point is the increased tendency of the steel to age and the related loss of toughness. It is true that this ageing will not take place so rapidly when the material is not brought above room temperature. I can imagine cases of use of the material for factory-produced components, however, in which the steel could be subjected to considerable heat, for instance when products are stove-enamelled or when they are plastic covered, a process during which they are brought to temperatures in the neighbourhood of 100° C. or even higher, as a result of which ageing is greatly accelerated. I should therefore like to point out the great responsibility that rests on the steelmaker in this connection to safeguard the user, who usually lacks specialized knowledge of metals, against disappointments in using drawn sections.

André FANJAT DE SAINT-FONT

The Problems of Factory Building

(Original text: French)

Factory building offers considerable scope to steel technicians. Quite a number of projects have been carried out for the erection of French-designed grid-plan or modular-type buildings which are just about models of their kind.

However, the framework of a factory building is not the whole story. The roofing of such factories is a problem for which quite a number of entirely satisfactory solutions have been devised, but there is still the problem of the walls, with all the incidentals such as air ducts and vents, and doors of different sizes. To my mind, all industrialized building should include dry walling, which of course involves cladding.

I should like to see the cladding manufacturers (enabled as they are by their roller bending machines to engage in mass production of sheet) turn out standard string courses, standard window sills, standard frames and trims for apertures, and even — why not? — standard doors. I feel certain that if they did, everyone would be satisfied at comparatively low cost, particularly the steel consumers, who would have a new market that had previously been closed to them for no real reason at all.

Otto JUNGBLUTH

The Problem of Cold Work-Hardening

(Original text: German)

May I reply to Mr. Maars on the problem of cold work-hardening. Obviously with the increase in strength due to cold-forming, -working, -upsetting, or -drawing there occurs a decrease in the elongation at fracture and in the malleability. This is experienced in many instances, including that of the motor car, where the bodywork is pressed and welded, but this on its own is not a determining factor in the decrease in elongation at fracture. Very many other factors play a part, a quite important one being, for example, the material thickness. I would not work-harden a 30-40 mm. thick drawn steel section and then weld it in its hardened state, but for steels less than 10 mm. thick, if they are adequately — I would repeat adequately — well manufactured, then utilization of the work-hardening process is perfectly feasible with present-day steel qualities. We have been gathering experience in this field for about fifteen years, and I can tell you that in these fifteen years I have never heard of a single case of brittle fracture when using drawn work-hardened steels of small thicknesses such as 4, 5 and 6 mm. in most cases. But with sections not cold formed, for instance hot-rolled sections, which have not been subsequently worked at all, I know of many cases of brittle fracture; so thickness does play a part here. I would say that it is also very evident that one might compare this work hardening to the administration of an antibiotic — that is, one cannot do the strengthening indiscriminately, but must do so with mature consideration, purely locally, correctly measuring it like a dose, so that the process is kept fully under control. Naturally, the strength can be increased by alloying — but that costs money; cold work hardening costs nothing because, for example, in a deep-drawing press or in a profile rolling machine, I can set up my apparatus, for example a profile rolling assembly, so that I obtain a higher strength section by physical, not chemical, means; within fixed limits, naturally.

I may draw attention to our concrete-reinforcing bars, on which the cold work-hardening process has been used for many years; there are indeed cold work-hardened reinforcing steels, which are also welded, for which a limit of not less than 8% elongation at fracture is laid down, at least in the German Standards.

If we laid down for cold-formed sections only, say, a 12% elongation at fracture after cold drawing, then I believe there would be no hesitation in using thin sections, given an appropriate steel. In any case, I can say that in my own practice I have not known a single case of brittle fracture in fifteen years, and we have welded all these sections.

Jean GALLIEN

The Problem of Co-ordination in the Industrialization Field

(Original text: French)

On the site, industrialization generally comes up against a different operational pattern, and the difficulties which arise from the simultaneous presence of industrial and of traditional craftsman-type processes.

Factory production is made difficult by the failure of the other trades to understand the requirements of industrial fabrication. The rhythm of production is broken owing to the numerous modifications requested by other enterprises which are unable to adapt themselves, by reason either of limited means or of their traditional structure, even though the designs were accepted when the contract was signed. The result is that costs are much higher than was estimated, and material is wasted.

This trouble could be with us for a long time, and co-ordination must be established to prevent chaos on our building sites.

Silvano PANZARASA

Prefabrication of Steel Building Components

(Original text: Italian)

As part of the Ina-Casa Plan, Cornigliano S.p.A. have embarked on a 1200-room housing programme for their employees at Prà, Genoa. The architects are G. and R. Ginatta. Two buildings in this complicated plan, (one of 14 floors and the other of nine,) involve the use of steel products and obviously have an experimental and comparative design function within a group of buildings of traditional reinforced-concrete structure. They are the only prototypes in the Ina-Casa programme, all the remainder being designed on traditional lines. Their main features are as follows:

(a) a rectangular plan covering 400 sq.m.;

(b) a load-carrying skeleton consisting of 22 vertical frames running transversely to the longitudinal axis of the building at intervals of 3.60 m.;

(c) the frames, comprising three stanchions spanned by the main floor-bearing girders;

(d) the external columns consisting of I-sections with a web depth of 180 mm., visible on the façade and connected to horizontal girders which are also exposed;

(e) the central internal tubular stanchions coated with vermiculite, and I-section floor joists;

(f) transvese bracings at the ends and in the centre of the building as well as longitudinal bracing in the centre, to withstand horizontal wind forces and asymmetrical loads;

(g) tubular members linking the frames at the joints between the principal beams and the upright, integrally connected with the supporting members of the staircase.

The load-bearing framework is anchored to the foundations by bolts built in the concrete plinths. The floors are of the traditional reinforced-tile type incorporating heating elements. A notable internal feature of the structure is the continuous beams with interrupted vertical members. The staircases are of steel, covered with traditional masonry-work, the steps being made of rubber-faced cold-formed zed sections. The façades consist of two fenestrated curtain walls and two semi-traditional windowless brick elevations with internal plasterboard panels.

The curtain walls, 5 cm. thick and weighing about 40 kg./sq.m., save about 400 sq.m. of floor space and substantially reduce the total load (by 280 and 440 metric tons in the nine-storey and 14-storey buildings respectively). They comprise:

(a) a framework;

(b) steel infilling panels with fibreglass insulation;

(c) window frames.

The framework consists of prefabricated sub-frames covering about 10 sq.m. and weighing 150 kg., with zed sections welded to the outside and neoprene gaskets on the inside for protection against rain, draughts and vibration. The thermal transmission factor K is 070 kcal/hr/sq.m°C. The infilling panels have an external Class A enamel finish fired at 800° C, the remaining external and internal steelwork being galvanised and painted. The external window frames are of cold-bent galvanised steel and are double-rebated, as are also all internal window and door frames. The internal wall facings are plasterboard and the sanitary equipment consists of prefabricated units complete with taps and fittings for each flat.

The foregoing account will serve to demonstrate the advantages of such prototypes, but it is equally clear that savings in cost can only be achieved if, instead of building prototypes, complete ranges of buildings of this kind are erected from cheap components produced in different parts of the country on a mass-production scale.

One of the essentials for economic construction is a regular supply of materials from factory to site, but at the moment this cannot be guaranteed. The example illustrated above, on the other hand, employs experimentally produced components and cannot be regarded as valid under normal conditions. In the field of low-cost housing, solutions of this kind cannot be looked upon as technically and financially sound until we can overcome the extra cost of providing items such as curtain walls and air-conditioning (essential if more comfortable summer and winter temperature conditions are to be ensured in the home) however satisfactory the technical and architectural results may be.

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2 Steel curtain walls, Series 3 - 2, U.I.S.A.A. monographs on steel applications.

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Hugo WAGNER

Standards for Structural Steelwork

(Original text: German)

The individual subjects discussed at the working party meetings show quite clearly that in every country everything is being done to encourage prefabrication or mass production — in short, standardization of structural steelwork.

I myself am a member of a working party of the Austrian Standards Bureau and know only too well that every country has its own Bureau or Institution for the production of Standards, and that, with the exception of Euronorm and ISO, there is no centralized Standards Institution. Euronorm is concerned mainly with steel qualities and ISO, in Committee No. 54, with overall construction. Up to now there has been no organization working for the unification of constructional steelwork standards appropriate for the European market. On the other hand, looking at the large European market, it is felt that the time has come for the entire constructional steelwork industry to prepare, with the exeption of statical terms, uniform Standards acceptable to every country.

It would be of especial value to firms concerned with mass production of structural steelwork for different countries to have to deal with only one, and not several, production programmes, and thereby be more efficient, cheaper and more competitive.

As a matter of fact a large European economic area is unthinkable without such an arrangement, so that this step is all-important for the development of the constructional steelwork industry. With increased efficiency and smaller stocks prices would be lower and deliveries quicker, and a preference for steel over other materials would not be dismissed offhand; the slogan "Steel for the World of Tomorrow" could then more easily be translated into fact.

I put forward as my solution that the High Authority of the European Coal and Steel Community should set up an advisory body made up of specialists from different countries and that this body should co-operate with the individual national Standards Bureaux or Institutions in order to bring to the European market a Standard as unified as possible, as for example a Euronorm — Stahl (Euronorm — Steel). The initial work for such a body would naturally be difficult because, at the moment, different standards exist in each country and these are changed or adjusted periodically. However it must be borne in mind that a start must be made sooner or later. It must also be decided what is to be included in the Standards and to what extent the individual Standards Authorities would have to be persuaded to co-operate. On this point, however, I am convinced that once these individual authorities understand why this co-operation is necessary they will not stand in the way.

To sum up, may I once again state that if we had unified Standards for structural steelwork in all European countries this would be a big step forward in the utilization of steel and could perhaps represent a major step towards a European Common Market, to which we all aspire.

Standardization further offers the following particular advantages:

(1) lower production costs, thanks to flowline production and saving on material;

(2) shorter production time and considerably reduced idle and set-up times;

(3) quicker turnover of operating capital;

(4) easier pre-planning; reduced impact of market and seasonal factors on procurements of materials and equipment, raising of capital, production and distribution; better market transparancy for the producer;

(5) appreciably simpler stockkeeping;

(6) better-quality products;

(7) more rational operation and administration, with lower overheads, higher productivity and smaller advertising expenditure.

Daniel Jean RÉVILLE

Comments of the Photographs concerning the Construction of Prefabricated Steel Flats

(Original text: French)

Here is an example of some prefabricated steel flats which have enjoyed a certain popularity and which have been occupied sufficiently long to enable the living and maintenance conditions to be assessed.

They consist of three and four-storey blocks. They were constructed for the personnel of a steel firm at a time when the construction of steel housing was not as developed as it is nowadays, with the object of demonstrating that flats so built could be satisfactory from the point of view both of initial cost and of comfort.

The small number of storeys justifies the use of a frame made up of cold-formed steel sections. The curtain walling is in steel, as is also most of the flooring and roofing.

Many flats of this type have been built: 150 in 1955, 900 between 1956 and 1958, and a further 112 in 1961/62. The last, moreover, were constructed under the second experimental scheme of E.C.S.C.

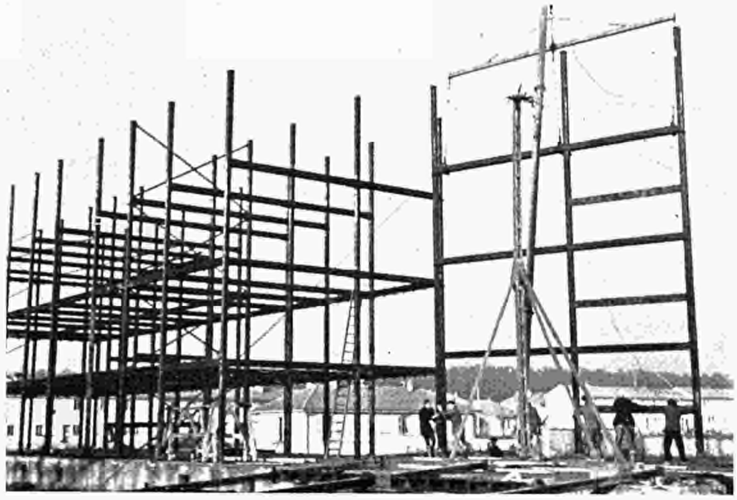
Although we are thinking in terms of prefabrication, with extensive preliminary work in the factory, there is as yet no question of mass-production. The members particularly of the frame must be selected for each scheme, and there is no stock from which one can draw members suitable for all structures. There is therefore considerable research to be carried out for each housing scheme, and perhaps special tooling to be designed, with the result that a fairly large number of dwellings must be involved to pay off this expense. Experience has shown that equality of cost with traditional construction, such as masonry, brickwork, concrete, can be achieved with about 150 dwellings. For a larger scheme, the initial cost will be less.

The second conclusion concerns habitability. Some of these dwellings have been occupied for 10 years or more, and from the start they have given satisfaction to the occupiers. No one has had cause to complain about sound or thermal insulation. They have also given satisfaction by reason of the excellent resistance to corrosion of the curtain walling, of galvanized sheet with one coat of paint applied in the works and one after erection. In comparing the cost of this form of construction with other types, one must bear in mind the need for occasional repainting. The buildings have maintained a completely satisfactory appearance.

The wall-cladding of many factories too has been carried out with curtain walling of the same type.

Description of photographs

1 — The start of the erection of a four-storey building.
2 — Details of the frame.
3 — Larger scale details of an assembly in course of erection.
4 — Fixing a window. All windows are delivered in their frames.
5 — View of a frame erected with floors and windows.
6 and 7 — Erection of wall cladding.
8 — Detail showing hooks for fixing the cladding.
9 — Finished structure as it appeared originally.
10 — Finished structure as it is today.



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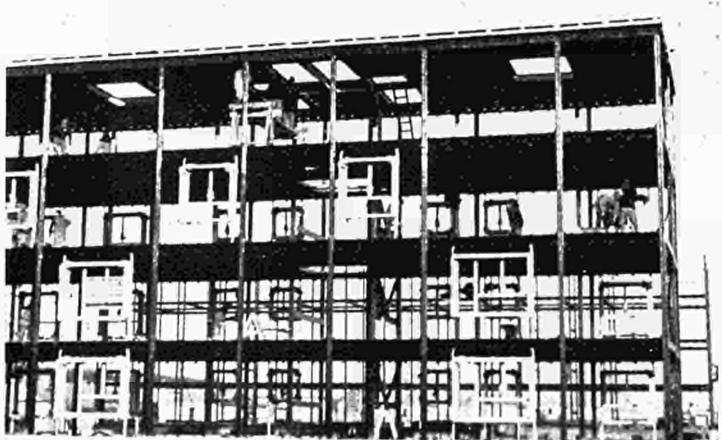

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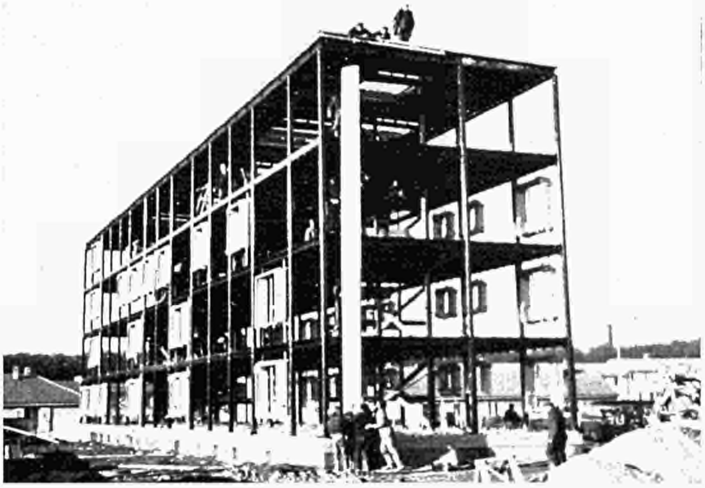
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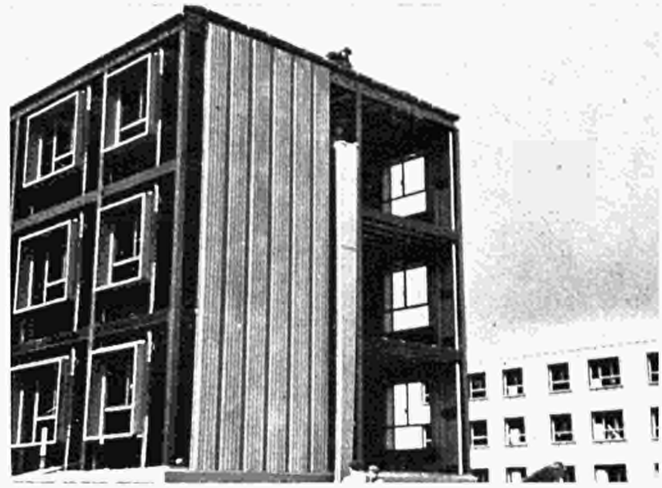
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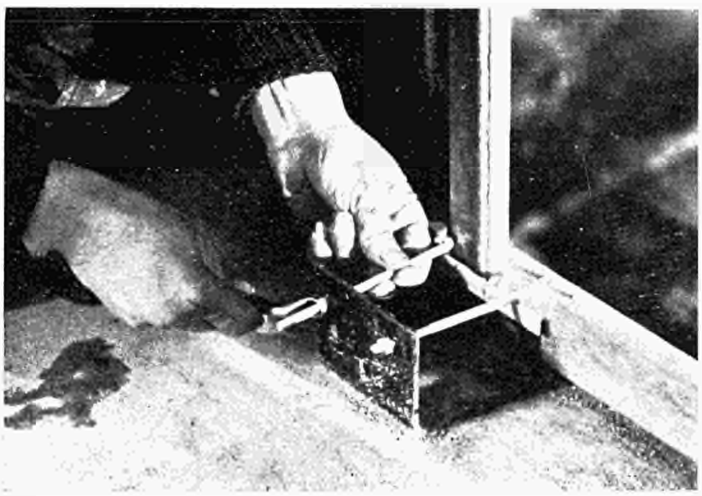
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Jacques BENDER

Prefabrication of Steel Building Components

(Oirigna! text: French)

I should like to tell the Working Party very briefly something of what we have been doing in the way of the industrialized production of steel building components.

In a converted motor works we have begun the mass-production of *all-steel* buildings. We mass-produce floors, walls, window-frames, partitions, ceilings, roofs and doors of galvanized sheet, all internal and external non-load-bearing clothing, and all the standard framework components. Each element is available in a very carefully-defined range. In a year's time, production schedules will be drawn up entirely on a punched-card basis.

By this means we are able to produce:

— a factory shed of 3,000 square metres in four days,

— a two-storey building of 2,600 square metres in a fortnight.

With long production runs it will be possible to reduce these times by 60%. Most of the operations are to be progressively automated.

We consider that an output of 20,000 square metres of building per month should be the minimum capacity for a prefabrication plant.

Findings

Steel construction should abandon methods in which fabrication is carried out as a craft and should apply industrialized processes, not only for industrial buildings, but also in housing and in the construction of administrative and public buildings.

— Combination of steel with other materials:

Steelwork contracting firms should offer a programme comprising, not only the structural framework, but also the internal and external walls, as also the floors and roofs, which would be in keeping with the considerable growth in the production of flat steel products. These firms should offer a complete service, even to the extent of providing the materials necessary for finishing the steel structures, *e.g.*, the concrete which has to be applied to the steel plates and, in other cases, insulating materials, special lacquers, plastics, etc. This is necessary in order that the use of steel shall fulfil the requirements as to insulation and surface finish and to fire protection regulations. Thus the steelwork contractor could offer the architect a structure completed to a far more advanced stage of finish, thereby minimizing the amount of skilled finishing work of various kinds to be carried out on site. In this way the architect could derive the best possible benefit from the advantages offered by structural steelwork in conjunction with prefabricated steel components such as roofs, walls and floors.

— The prefabrication of steel components presents a large number of general problems. It is proposed that the High Authority should take an interest in all research in this field.

— The architect's training:

The present training of architects is often directed towards construction in concrete, at least in some countries. It would be desirable that steel construction be given the place to which it is entitled in the training of architects.

WORKING PARTY V:

Prefabricated Standard Buildings and Mass Production of Building Units

Chairman:

Pierre VAGO

Rapporteurs:

Jan SITTIG

Walter HENN

Working Party V dealt mainly with questions relating to the industrialization of building methods (residential and functional buildings).

It noted the importance for this purpose of standardization of building components, fundamental and applied research, and revision of present building regulations and practices.

Changes on the scale required would, it was considered, involve the introduction in architectural, engineering and technical colleges of theoretical techniques and practical instruction in industrialized building.

The Working Party proposed that a European Institute of Industrialized Architecture be set up to co-ordinate the various steps to be taken concerning standardization, research, the framing of regulations, and training organization.

Most of the speakers, whether from the engineering, the industrial or the architectural side, considered steel to be particularly suited for industrialization purposes.

Jan SITTIG

Industrialized Construction and Steel

(Original text: Dutch)

Industrialization necessary

In the last hundred years the production of goods has undergone a truly revolutionary change as the result of a complex process which may be summed up as transition from production by artisans' methods to industrial production. In this connection the most striking feature is the tremendous increase in productivity of labour, in the quantitative sense, due to mechanization and automation. In addition, there has been a vast improvement in quality, which would have been impossible without the advent of the machine. Finally, industrialization has laid the foundation for the emergence of entirely new products which, in all their diversity, high quality and low price, are the manifestation of the prosperity in which the Western World lives.

Amid this prosperity the construction industry forms a notable island of poverty. The increase in the productivity of labour in building is slow; mechanization and automation are still only in their early stages, and the quality of building is deteriorating rather than improving.

Twenty years after the end of the Second World War we have not yet succeeded in raising the means of satisfying the need for housing accommodation — one of the first necessities of life — to a quantitatively and qualitatively acceptable level.

The discrepancy between the building output and that of other commodities can be removed only by speeding up the industrialization of building.

In order to combat the lagging-behind of the construction industry, it is necessary to know the causes of this.

In our opinion, the primary reason why the construction industry is finding it so difficult to break away from the "artisan" stage is that buildings are "immovables", permanently fixed to the ground and therefore unsuitable for manufacture in a factory.

The second cause is that, more particularly the building of houses, the various functions — initiative, design, production, use, and maintenance — are spread out over a large number of persons and organizations. Consequently, there is a narrowing of interests, which state of affairs leads to typical sub-optimalisation.

Finally, the long service life of buildings constitutes the major reason for the inability of the construction industry to move more rapidly into modern industrial development. The reason for this is that industrial development is powerfully stimulated by the continuous interaction of "thinking" and "doing", which is the consequence of the "feed-back" of the experience gained in production to the design and of the experience gained in use to production and design. [1, 2]

Every industry develops a "control system" of this kind, and it is obvious that, in addition to the system whereby feed-back is effected, it is more particularly the speed of feed-back which determines the value of such a control system. The long service lives of buildings and, indeed, also the long production time involved ("long" in relation to the lives of the users, designers, builders, etc.) have resulted in the failure of natural feed-back to develop. Industrialization of building will, for this reason, succeed, only if co-operation between research and production is achieved in the form of a development-cycle centred upon accelerated and well-directed feed-back of information.

The stages of industrialization

When one speaks of industrialization, the engineer thinks of mechanization, the economist thinks of a particular cost-price structure, the designer of industrial design, the organizer of the continuity of the production process, and the business man of sales methods suitable for the mass-produced article.

In order to avoid losing our way in this multiplicity of aspects, we have endeavoured to establish a schematic relationship between what we consider to be essential stages of the industrialization of building, in which scheme the principal activities associated with the various stages, and the results thereof, are indicated (Fig. 1).

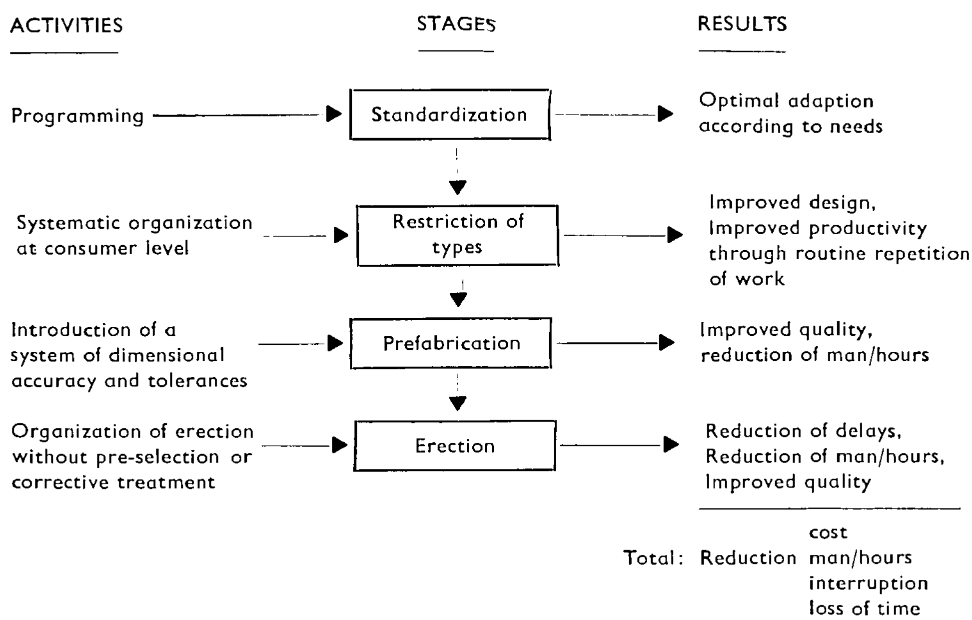

Fig. 1

The first stage is the standardization of types of buildings, components, details, etc. Standardization means that, from the infinite number of theoretical possibilities, a limited number of solutions are finally chosen for actual application.

Standardization is the starting point of industrialization. An industrialization process that suffers from a wrong choice of starting point will never be able to yield its full benefit. Hence it follows that it is necessary to evolve a theory of good — preferably optimal — standardization, as will be out-lined in a subsequent section of this paper.

The next stage of industrialization is the restriction of types. In a sense, this is the commercial realization of standardization, which in itself is merely a programmatic exercise on paper. Industrial production presupposes manufacture in quantity, *i.e.*, mass production, and this—again because of the long service lives of buildings—can be achieved only by systematic organization at the consumer end (*e.g.* by co-operation on something more than a municipal basis in drawing up and carrying out housing construction schemes) and at the producer end (by co-ordination of projection, planning, production and marketing).

Obviously, one consequence of such mass production in the sense of large series of identical products is that it becomes practicable to devote more time and money to design. Furthermore, the productivity of labour is enhanced as a result of the routine repetition of identical operations, even without the benefit of mechanization and automation.

The last-mentioned features are introduced in the next stage, namely, that of prefabrication. By this is implied that part of the constructional activities, more particularly the manufacture of the components of the buildings, is no longer carried out on the site, but in the factory. This in turn implies a more advanced division of labour, with the possibility of mechanization, better quality of the structural components, and a reduction in cost.

However, as compared with the "artisan" method of construction, prefabrication gives rise to new problems, particularly in the sphere of transport and, above all, dimensional accuracy and tolerances. With "artisan" construction, those parts of a building which have already been completed will determine the dimensional requirements of other components which must be fitted to them. For example, the space between two window frames will be filled up with brickwork. Strictly speaking, this kind of construction, based on adapting each additional part of the structure to the part already in position, could be carried out without the aid of any sort of numerical measuring equipment.

With prefabrication on the other hand, an "opening" is manufactured in one factory and the "infilling" is manufactured in another. When a certain number of each of these two types of component have been delivered to the building site, they should be capable of being assembled together without any pre-selection and without requiring any subsequent treatment to make them fit. In fact, this was the problem that faced mechanical engineers fifty years ago and which, in that branch of engineering, was solved by the introduction of systems of tolerances and fits, with the associated methods of measurement. A similar development is to be observed in the field of building construction, though this development is still only in its early stages.

The logical consequence of prefabrication (and the development of a good system of tolerances and fits) is the fourth and final stage of industrialization, namely, erection. The activities on the building site are then confined to the assembly and fitting together of the structural components prepared elsewhere, while subsequent corrective treatment is obviated in all but very exceptional cases. Erection can very largely be mechanized, and automation of the erection operations becomes a real possibility.

The results of industrialization can be summarized as the attainment of higher quality at reduced cost, in less time.

An example of programming

Industrialization starts with standardization, which, if it is to be efficient, is the result of sound programming.

The problems that such programming entails with regard to certain types of building will be discussed in this section with reference to an example : the programming of industrially manufactured (*i.e.* prefabricated and erected) shed-type buildings for industrial purposes.

Subjects for this programming will primarily be the dimensions and, furthermore, the construction of the supporting structure, walls, windows, doors and roof.

With these structural details however we already move into the sphere of design. We shall draw the demarcation between programme and design at that point where mere words and figures no longer suffice as descriptive indications and where, therefore, the design — in the sense of shape and form — begins to become essential.

In our example we shall confine ourselves to the dimensions of the industrially produced shed-type buildings and, more particularly, to the spans.

Every system of standardization has advantages and disadvantages. We need not dwell on the advantages, since we have already demonstrated that standardization is the prerequisite that must be fulfilled if industrialization is to become possible. Prefabrication is inconceivable without standardization, and any one wishing to undertake industrialized building will have to adopt standardization as a matter of course.

The need and the desirability of standardization must not, however, blind us to the disadvantages thereof. Thus, with the standardized product, generally speaking, the needs cannot be so completely satisfied as with the "artisan" system, which can be varied to suit individual requirements.

An industrial undertaking needing a shed-type factory building in which machinery, mechanical handling equipment, storage racks, etc. have to be installed will, as the result of a sound functional investigation, come to the conclusion that a span of, say, 48 ft. 6 in. (14.8 m.) is required. As a rule, the precise desired size cannot be found in the catalogue of prefabricated components, *e.g.*, roof trusses. In that case it is necessary to adopt a different, and usually higher, value for the span, *i.e.*, one which is catered for in the programme of standardization. Because of this there is an "adaptation loss", which is greater as the gap between what is desired and what is available increases.

Programming in the first place calls for an understanding of the nature and magnitude of the adaptation loss. Structural calculations — based on reasonable assumptions — show that the weight (and also the cost) of a span is proportional to the 7/3 power of its span length. However, in determining the adaptation loss it should also be taken into account that a larger span than is required does mean that extra width becomes available in the building, even though this extra width must be rated at a relatively lower value.

Let α denote the ratio between the standardized and the desired span ($\alpha \geq 1$). For mathematical convenience the additional space can be given a value rating represented by the factor $\sqrt[3]{\alpha} - 1$. The adaptation loss, expressed as an extra charge over and above the cost that would be incurred if the desired span coincided with a standardized span, will then be $\alpha^2 - 1$.

This adaptation loss for spans ranging from 16 ft. 5 in. (5 m.) to 131 ft. (40 m.) is represented in figure 2.

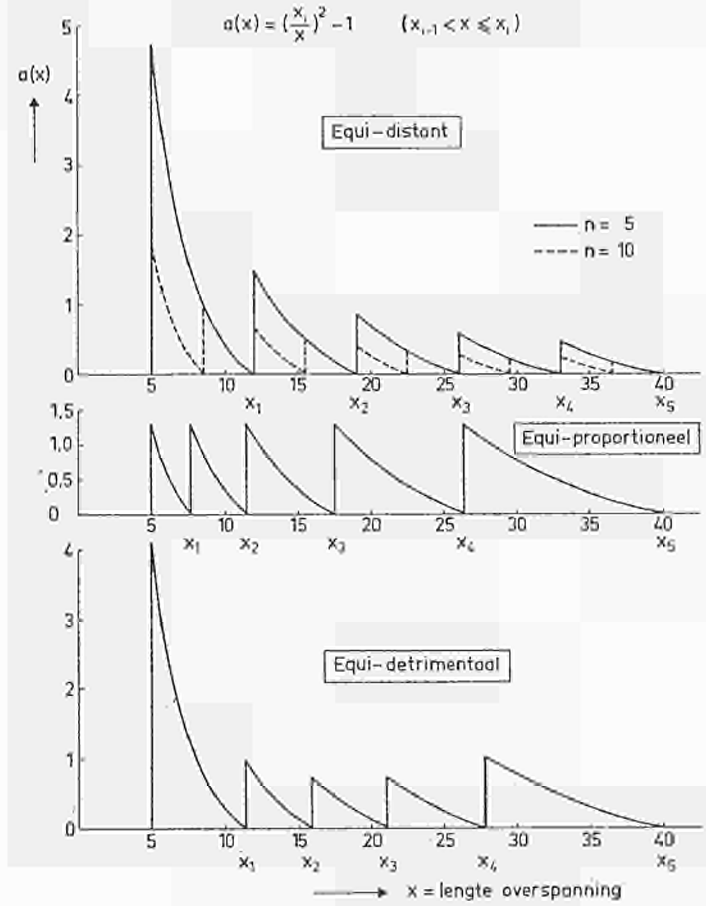

Fig. 2

The form of the adaptation loss function depends on the number of types to be standardized and on the situation of these types. The latter aspect could be called the formation principle of the standardization concerned. In practice, an arithmetical progression is often chosen for the purpose, *i.e.*, equal distances between the consecutive standardized types. Alternatively, a geometrical progression is sometimes preferred, *i.e.*, equal ratios between the consecutive standardized types. Of course, there is an infinite number of other principles on which the standardized series could conceivably be formed.

In addition to an equidistant (arithmetical) and an equiproportional (geometrical) formation, figure 2 includes an "equidetrimental" formation, *i.e.*, one in which the adaptation loss is the same for each standardized type.

From the diagram it appears that an increase in the number of types will result in a reduction of the adaptation loss. The question as to the choice of the number of types will be further considered later on.

Clearly, a knowledge of the adaptation loss function alone does not provide complete information on the adaptation loss itself. The latter will also—and, indeed, very considerably—depend on the nature and form of the requirements distribution.

With regard to the problem of the standardization of prefabricated factory buildings, reference has been made to a "requirements distribution" investigation by Neufert.[3] The requirements distribution can be expressed by a gamma function with a mean value of 20 m. and a standard deviation of 8.2 m.

Figure 3 shows this requirements distribution and also the equidetrimental adaptation loss function for five standardized types.

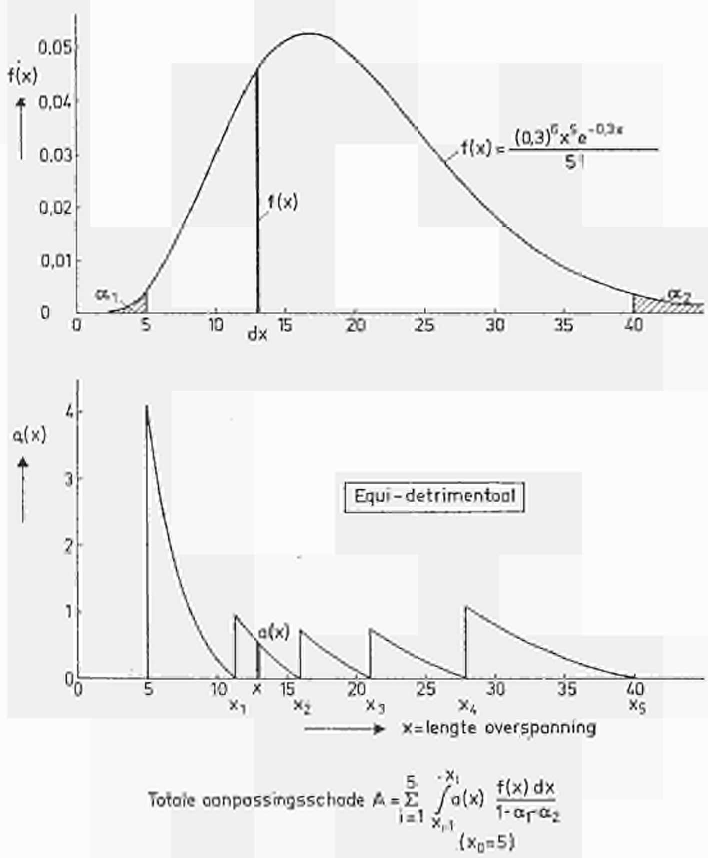

Fig. 3

The total adaptation loss is obtained by multiplying the adaptation loss a(x) associated with each span by the probability of the occurrence of this span (as the desired span) f(x)dx, and by integration, or summation, of this product over the selected range of requirements distribution, *i.e.*, from 5 m. to 40 m. in the case under present consideration.

In this way we obtain the average adaptation loss — expressed as a fraction of the cost that would be incurred if there were no adaptation loss, that is to say, if the requirement, or actual need, coincided with the standardized value — which could be expected to occur with five standardized types located as indicated by x_1 to x_5 in figure 3.

The magnitude of this average adaptation loss is, in the first place, dependent upon the number of standardized types and, secondly, upon the formation method adopted. As regards the latter aspect, it would appear in this example that the equidistant (arithmetical) progression is the least favourable and the equidetrimental progression the most favourable of the formation methods investigated. This does not mean to say that this last-mentioned progression, or series, will indeed give the least adaptation loss, but it does appear unlikely that any further major improvement can be achieved.

A similar calculation will also have to be carried out for other numbers of standardized types, so that the minimum adaptation loss is obtained (as shown in figure 4) as a function of the number of types to be standardized*.

As already stated, this minimum adaptation loss decreases if the number of types is increased. It would therefore seem to be desirable to make the number of standardized types very large. On the other hand, however, with an increasing number of types the cost of manufacture and distribution will rise because of the reduction of the "series length". The optimum solution will evidently be so located that the sum of the adaptation loss and of the manufacturing and distribution costs attains a minimum.

In figure 4 the additional manufacturing and distribution costs (over and above the minimum costs that would arise if only one type were produced) have been taken as proportional to the fifth-power root of the number of types to be produced (this assumption is based on unpublished calculations for comparable cases).

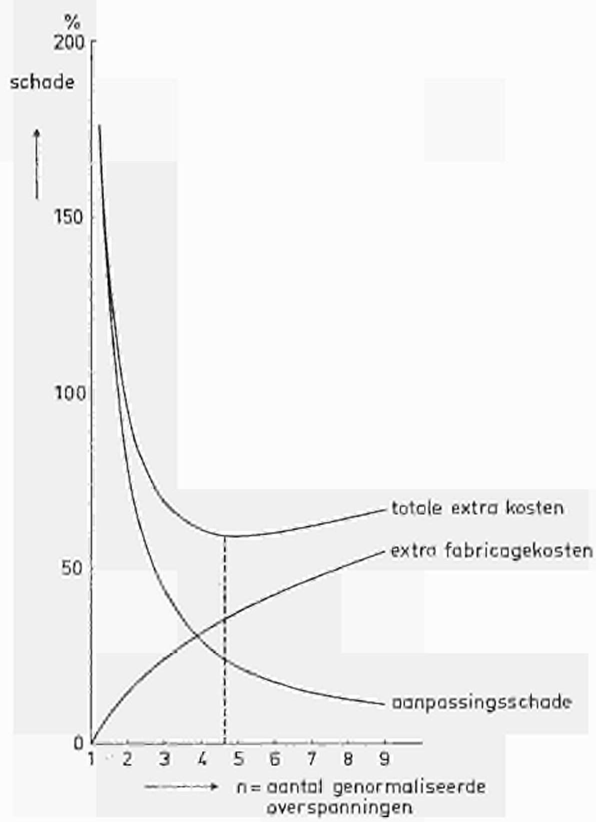

Fig. 4

* It has also been assumed that the quadratic adaptation loss relates only to one-half of the cost of purchase of the factory building.

Also indicated in the same diagram is the line for the total cost, which attains its minimum for $n = 5$. On the assumptions made, this represents the optimum number of types that should be included in the programme for the standardization of spans for shed-type factory buildings.

In this (optimum) programme we have:

the average adaptation loss = ...	21%
the additional manufacturing and distribution costs = ...	38%
the total cost = ...	59%

of the "basic cost".

This basic cost is merely of theoretical significance; it would be incurred if the entire requirements distribution were concentrated into a single point, so that the entire need could be satisfied by one type, without adaptation loss.

The minimum of the total cost in figure 4 is represented by a flat range of the diagram, as is often the case in optimalisation problems of this kind. Minor variations in the number of types to be standardized therefore do not greatly affect the total cost, and in this connection it is to be noted that an increase in the number has a less pronounced effect than a reduction thereof has.

Of greater importance than the choice of the correct number of types for standardization is the choice of a correct formation of the series of standardized values. The following summary shows four potentially suitable series for which, in our example, the average adaptation loss has been calculated:

Formation	Spans to be standardized (in m.)	Av. adapt. loss
equidistant	(5) 12 — 19 — 26 — 33 — 40	**25%**
equifrequent	(5) 12.9 — 16.8 — 20.7 — 25.7 — 40	**24%**
equiproportional	(5) 11.6 — 15.8 — 21.5 — 29.3 — 40	**22%**
equidetrimental	(5) 11.3 — 15.9 — 21.0 — 27.8 — 40	**21%**

It must be pointed out once again that this result is bound up with the chosen approximation of the requirements distribution, with the chosen adaptation loss function and with the chosen approximation in the relation between the number of types to be produced and the manufacturing and distribution costs. In order to arrive at an optimum programme in a specific actual case, these three functions would have to be studied with greater precision than has been done in the present example, the sole object of which was to demonstrate the programming principles.

Steel

So far, the examination of the present subject has been in such general terms that there has been no necessity to refer to specific construction materials. We must now consider the question as to whether, in the process of industrialization of building as has been very briefly discussed in the foregoing sections, steel will be able to play a special part. In our opinion it will for the following reasons:

(a) Acceptance and adoption of the industrialization principle, which in our scheme comprises the successive steps: standardization, restriction of types, prefabrication, erection, can be expected sooner from steel manufacturers (rolling mills and steel fabrication works) than from the manufacturers of other construction materials. This is because steel manufacturers already think in industrial terms; they have already experienced a similar cycle of industrialization in other fields and have actively promoted it. For this reason they will have less psychological difficulty in effecting the industrialization of building than will the manufacturers of conventional materials, to whom the idea of industrialization is novel and alien and therefore appears objectionable.

We have put this factor first because, in a different sphere, we found, to our loss, that the psychological resistance to industrialization can easily prevail over the economic and technical advantages thereof.

(b) Steel is exceptionally well suited for the construction of adaptable buildings. This is an important point if we take a "dynamic" view of the need for buildings, *i.e.*, with regard to the changing needs with the passage of time. By using steel it is possible to minimize the adaptation loss which arises from the discrepancy between, on the one hand, the relatively rapidly changing functional requirements that buildings must fulfil and, on the other hand, the long service lives of buildings which are difficult to convert so as to make them suitable to fulfil such requirements. As a final consequence, in this connection, the ease with which structural steelwork can be dismantled and removed calls for consideration.

(c) In our model of industrialization the problem of tolerances and fits plays a decisive part. It is certain that steel lends itself better to accurate dimensioning than do other materials, and in this connection we must bear in mind the invariability of the initially established shape and dimensions with the passage of time.

For example, if it is desired, in a particular section of the construction of residential buildings, to undertake the industrial manufacture and erection of door and window frames, then the scope for doing this if steel frames are used for the purpose does indeed exist, as has also been established by an investigation instituted by the High Autority of the E.C.S.C.[4] Whether this would also be possible with wooden door and window frames may well be doubted.

(d) Steel as a construction material has the advantage of possessing great strength. This means that, for comparable loads, steel structures can be of much lighter construction than, for instance, structures built in concrete and, more particularly, that the volume of the structural components—which from the functional point of view is to be regarded as wasted space—is very small. The lower weight in turn means lighter and cheaper foundations; the smaller volume results in less space being wasted, and in these days of increasing prosperity, space is a commodity which is becoming more and more precious.

(e) Lighter construction in steel is made possible, not only by the great strength of this material, but also by its relatively slight variation of strength.

Large variations in the strength of a constructional material, on the one hand, and in the magnitude of the maximum load to which a particular structural component is subjected on the other necessitate high factors of safety, which are rightly to be regarded as "factors of ignorance".

For example, designers may speak of five-fold or eight-fold safety. In this connection it is of interest to cite Hagen, a German author:

> "To provide a multiple factor of safety is a pointless waste of money. It is therefore surprising that public authorities and companies which have buildings erected are prepared to approve the cost in cases where schemes involve ten- and twenty-fold structural safety, *i.e.*, where it is admitted that the cost of construction has been increased above the actual need in very nearly the same ratio. When a tailor says he needs three yards of material to make a coat, no one would dream of giving him thirty or sixty yards just to be on the safe side."[5]

If we remember that this was written a hundred years ago, then there are indeed grounds for pessimism with regard to the rate at which scientific knowledge has its effect on actual practice.

A great variation in strength properties means that one is faced with a high degree of uncertainty in answering the question as to what strength an individual structural component should have. By and large, the same is true of the maximum load to which any particular component will be subjected during its normal working life.

This uncertainty leads to considerable wastage. This is because the designer wishes to safeguard himself against situations in which the load will exceed the strength (or the load-carrying), so that failure will occur; hence he must adopt the most unfavourable (lowest) value of the strength and the most unfavourable (highest) value of the load.

This line of thought is represented in figure 5. In this diagram a material having a high coefficient of variation in its strength properties, namely, 20%, has been associated with a loading situation which likewise has a coefficient of variation of 20%. Avoidance of failure, *i.e.*, reduction of the probability that a structural component taken at random will be too weak to carry a load likewise taken at random to an acceptable value of, say, 1:10,000, means that for an average load of 100 the average strength must be established at 425. This therefore corresponds to a safety factor of about 4*. Now if we consider a construction material whose strength properties have a coefficient of variation of only 5%, while the variation in the load remains unchanged, the same degree of safety can be obtained with only half the average strength, *i.e.*, in that case the safety factor can be reduced to 2.

If a better knowledge of the loading situations likely to occur could enable the coefficient of variation of the load to be halved, the average strength of the components could be further reduced, namely, from about 200 to about 150, so that eventually a safety factor of 1.5 would emerge. It is therefore evident that, as a result of better control over the material properties (manifesting itself in a lower coefficient of variation of the strength), tremendous savings in material can be effected without loss of structural safety.

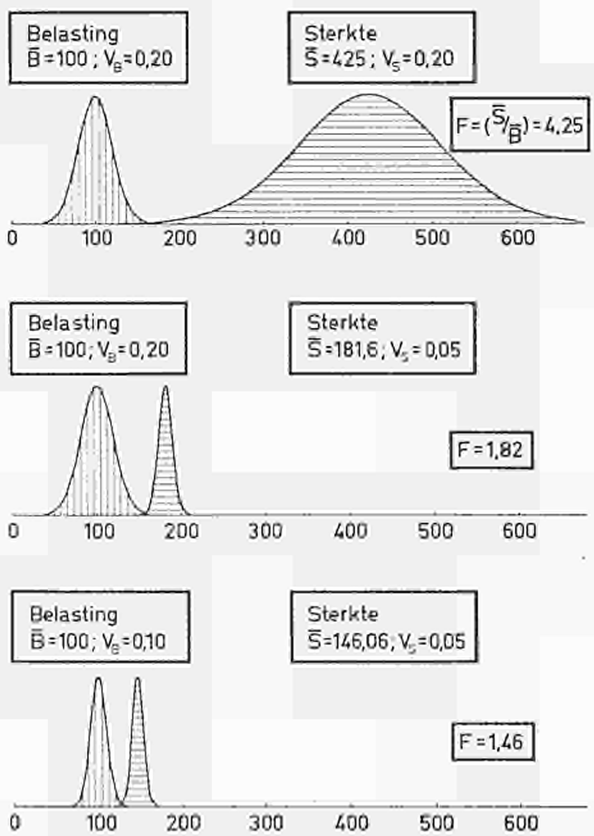

Fig. 5

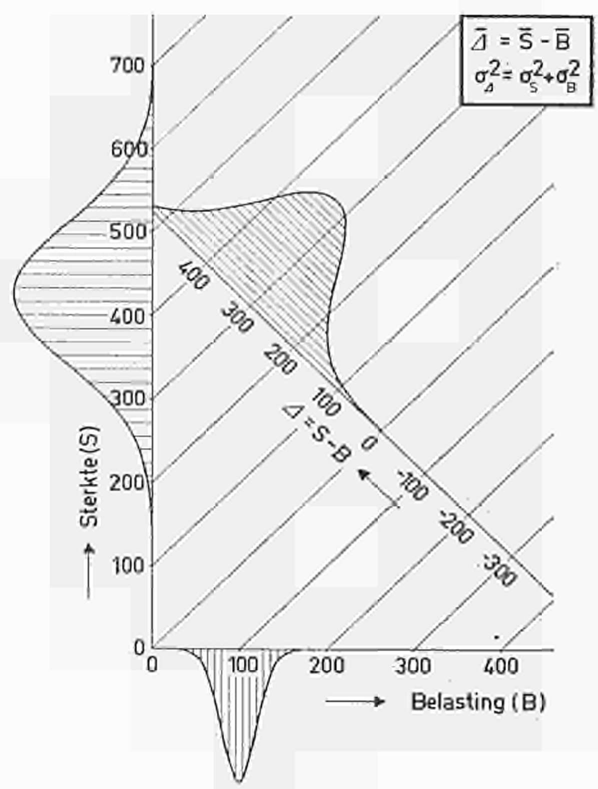

Fig. 6

* The calculation is performed by the introduction of a new variable $\Delta = S - B$, which may be called the "surplus strength". The probability distribution of Δ can be deduced from the known distributions of S and B (see Fig. 6). The risk of failure is the risk that Δ becomes negative.

The examples chosen are real ones. The coefficient of variation of the compressive strength of concrete cast *in situ* is, in fact, of the order of 20%, while the corresponding figure for good factory-made concrete (precast concrete) is of the order of 10%. The compressive strength of concrete building blocks has a coefficient of variation ranging from 11 to 30% (based on data issued by *Stichting Ratiobouw*, Rotterdam).

The compressive strength of timber has a coefficient of variation of between 16 and 20% for very carefully selected samples having the greatest possible homogeneity.

We have not been able to find any data on the variations in the compressive strength of steel in the literature. However, from statements made by experts to the effect that the specified tensile strength tolerances (about 20% tolerance range) can be adopted, though not easily, it is possible to calculate a value of 4 to 5% for the coefficient of variation.

Figure 7 shows a nomogram from which we can see how the factor of safety is dependent upon the coefficients of variation of the strength and of the maximum load, respectively. Pioneer work on the statistical evaluation of safety factors has been done by Committee W 23 of the International Council for Building Research Studies and Documentation (CIB) headed by the late Mr. E. Torroja.[6]

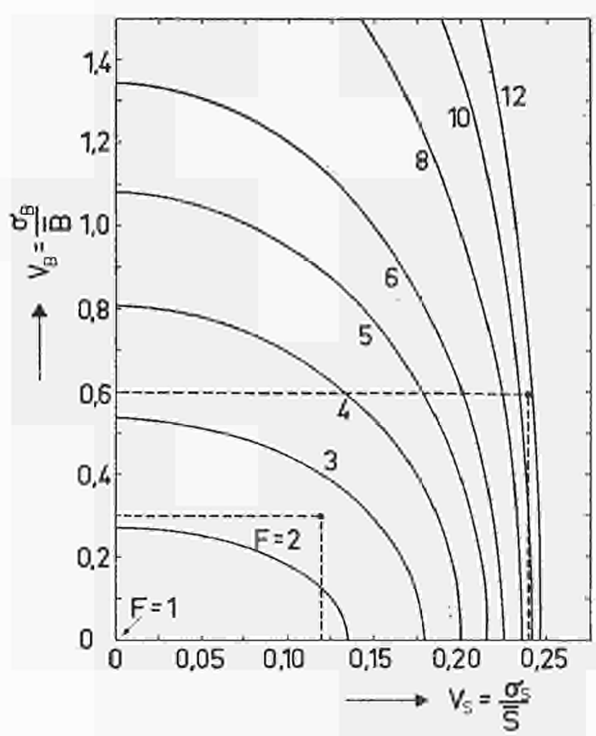

Fig. 7

Further, in its quest for high-tensile structural steels—which in turn will enable the weight of structures to be further reduced—steel technology would do well to devote to the problem of the variations in steel quality the attention that it merits in view of what has been said in the foregoing.

Problems yet to be solved

In the preceding section, properties of structural steel have been mentioned, which indicate that steel has a particularly important part to play in the process of industrialization. Of course, steel also has properties that may at times mitigate against its application:

(a) Steel is more sensitive to corrosion than are some other construction materials, so that in many cases it is necessary to take special precautions, either once only, as an initial measure, or repeatedly, in the form of maintenance.

(b) The insulating properties of steel are unfavourable, both with regard to heat and to sound. This, too, calls for special precautions.

(c) The strength of steel deteriorates as soon as a certain temperature is exceeded. Fire precautions therefore necessitate the application of special measures in many cases.

(d) The above-mentioned disadvantages of steel as a structural material can be reduced or obviated, but this invariably involves special precautions and therefore extra expense. Because of this, steel is in some situations more expensive than rival materials.

In a case where a particular building would cost more to build in steel than in some other material, it has been, and still is, usual to decide in favour of that rival material. This calls for two comments.

In the first place, the initial cost is, as a rule, not a correct criterion of choice. A better criterion is the concept "all-in cost", which comprises, not only the initial cost, but also the running costs (up to and including the eventual demolition of the building).[7]

On account of the adaptability of steel as a construction material it will often occur that the initial cost of the steel structure is, indeed, higher, but that the "all-in cost" is lower than that in the rival materials. Failure to look beyond the initial cost entails a risk of self-deception.

Secondly, well-founded objections can be raised to the use of cost (either as initial cost or "all-in cost") as the only criterion of choice. Mathematical programming, which has been applied so successfully in recent years, shows that there are indeed situations where the main obstacle to the execution of any particular production programme or building programme is not the lack of capital but, for example, the dearth of labour. In a situation of this kind the best strategy consists, not in seeking the cheapest solution in terms of money, but the cheapest in terms of man-hours. It is here, in particular, that steel—thanks to its industrialized fabrication and erection—shows up so favourably.[8]

Conclusion

Building construction by "artisan" methods involves heavy physical labour performed in wind and weather by unsuitably dressed workmen who carry out poorly organized operations with primitive tools, resulting in a low-grade building that takes a long time to build and requires a large labour force.

Once industrialization of building becomes a fact, it would be possible to conceive a building site where white-coated gentlemen sit at switch-boards controlling the mechanized handling equipment which smoothly fits together the factory-made structural components. The use of such tools as the plane and the saw would not be tolerated.

Buildings of optimum quality will spring up like mushrooms and on taking a closer view we shall perceive that these buildings—and indeed the whole industrialization process—are supported by steel.[9]

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Walter HENN

Prefabricated Standard Buildings and Mass-Production of Building Units

(Original text: German)

In his excellent paper Herr Sittig gave us an excellent review of the problems connected with the theme of our Working Party. I would like to add supplementary information based on my personal views and on my experience as an architect faced daily with these problems.

Unfortunately the problems of prefabrication, series production, industrialization—and even more if you take into account one constructional material alone, steel—are too frequently regarded as special problems which concern only a few experts.

In this I see the first danger linked with the theme of our Working Party: the view taken is much too limited, too one-sided. Why, in fact, do we discuss the somewhat dry subject: "Prefabricated standard buildings and mass production of building units"? At first sight this formulation is absolutely incapable of understanding; it actually appears as something which is only for specialists.

I am of a different opinion. Prefabrication only in building has admittedly achieved quite considerable successes, but it could not reach beyond a certain relatively restricted sphere of application. In building, mass production has its limits right from the outset, because the law of large series, which we know from other branches of engineering and industry, is not directly applicable to building.

By industrialization of building we can understand everything which we wish to understand. Industrialization of building is logical and purposeful only if the duty which it is to fulfil is fully evident and also if the necessary conditions are viewed from every angle. However, we are still a long way from this.

What then do we mean by: "Prefabricated standard buildings and mass production of building units"?

This signifies purely: What must happen; who has to get together with whom; what has to be co-ordinated, to enable building to be rationalized, so that the enormous amount of building with which we are more or less helplessly confronted in each country can be coped with more rapidly, better and more economically than in the past?

It is obvious that the production of 1,000 cars per day cannot be achieved by individual manufacture by craftsmen, and that new production methods are necessary. This problem is tackled sensibly everywhere and dealt with on the basis of economic considerations. The methods employed in the various countries differ only slightly from each other.

It is accepted that the building of 1,000 houses can no longer be accomplished using yesterday's bricklayers and carpenters. However, the way in which 1,000 dwellings should best be built today appears to be a problem of ideology or philosophy.

Why this discrepancy? Where is the difference between the manufacture of 1,000 cars and the erection of 1,000 homes?

First, quite a simple distinction: one can produce 1,000, 10,000, or 100,000 cars without there being a necessity to vary the dimensions of the cars. The purchaser needs to select only the colour of the bodywork and the seats to accord with his own taste. It is however not possible to make 100,000 dwellings, all of which have the same dimensions and the same plan.

If two houses are to be erected in a road, one house on the right of the road and the other on the left, then these two houses cannot be identical because if, for example, the road proceeds from east to west, the entrance to one house is from the north and to the other from the south. This apparently minor difference necessitates two different plans. To a much greater extent than in other engineering products, every building is influenced by its environment. Links with traffic, orientation in accordance with the compass, position on the level or on a slope; all these factors influence the house in its structure.

One would need to live in Utopia to wish to erect 1,000 identical houses, because there are not 1,000 sites available which present the same conditions.

Yet a further difference : A childless married couple and a couple with four children can quite easily use the same type of car — let us say the much-advertised "medium priced" car. For a family with four children the car is admittedly somewhat constricted, but, of course, it is only used for a few hours at a time.

A married couple with four children however, require a radically different dwelling than a childless couple.

These differences can also arise from differences in profession. A factory worker requires a different type of dwelling than a doctor. Both may however, perhaps have the same make of car.

The result of these brief considerations: it is possible to mass produce a car, but not a house. The mass production of complete houses is impossible, this can only be done with components; *i.e.* a standard unit system which permits the largest possible number of variations.

It is obvious that the conclusion is trivial, but the question as to how this standard unit system should be organized leads right to the heart of the problem. It is possible to agree to concentrate on a few complete basic elements having the same dimensions and to link these one to another in different series. Or else the structure of a building can be standardized, the dimensions of its individual parts being altered.

I would like to term the first system the "closed" system and the second the "open" system.

The use of a few photographs will clarify things more easily than is possible by a verbal description. Regardless of which standard unit system is employed, the pre-requisite for any system is a module.

We thus arrive at a requirement which was already discussed and the importance of which cannot be stressed sufficiently, precisely at this Conference. We require a European module for building. Only then will the conditions be created for the prefabrication and for mass production in building.

The module alone is not everything, however; with the module we also have the problems of tolerances and fit.

As to which module we specify, this is solely a problem of willingness to agree. There is actually no final proof that one module, say the octameter system with a basic dimension of 12.5 cm, is better than the decade system with the basic dimension of 10 cm. It is merely a question of agreement.

In the case of tolerances however, the situation is different. These depend on the material, manufacture, design and the assembly process. When deciding upon tolerances one is confronted with difficulties. the majority of which have not even been considered. However, there is all the more need for this problem too to be considered. I regard it as essential that our Working Party should draw the attention of the High Authority with all urgency to the necessity for having a decreed module and for deciding tolerances. I must now make a few general remarks, using once more for this purpose the comparison between "car" and "house".

The appearance of a cylinder head of a car is a question of suitability and economics; in short purely an engineering and economic problem.

However, the appearance of the columns in or around a building is not merely a question of engineering, of stability, suitable material and economics—but instead somewhat more. This "somewhat more" can be described and understood only with difficulty, including as it does beauty and symbolism.

This is why such great confusion exists in building. One person sees merely a technical problem in building; others rush to the barricades to defend architecture. In reality, building is even more complex than it usually appears to be.

It is easily seen and clearly appreciated that, whilst a building is being planned or is still in the course of construction, the decisive part is played by engineering and economic problems. As soon however, as a building is finished and occupied, nobody asks any further questions concerning building costs or how the building is put together. Instead, overnight, the only assessment of it is, whether it is beautiful or not. Suddenly, emotional values play the decisive role.

We should not take amiss the fact that these emotional criteria are applied initially to the finished structure, because everything emotional is similarly a direct reaction, the finished structure being the prerequisite. Only when I can enter a house are my feelings ready to react, and not before.

However, we should not ignore this emotional response during our considerations. In the case of the car it is taken for granted that it consists merely of steel and plastic. But a house consisting solely of steel and plastic would be rejected by the majority.

The average person believes that a room would lose its comfort if the walls were made of steel. At exhibitions of prefabricated houses it is noticeable how visitors knock the walls to establish the material from which they are made. Whether this is solely a question of habit, or whether lower layers of consciousness respond during this sensation reaction, would be worth an investigation. Perhaps, in this rejection of certain constructional materials for house building, a part is also played by the fact that inside our four walls we wish to escape from "technique".

Old petroleum lamps now command a high price. Antique dealers' shops are sprouting like mushrooms. These are the reaction to our engineering age.

Furthermore, in many things which we require and which surround us we no longer have any true freedom of choice. Basically we all drive the same car, we use the same refrigerator, the same washing machine, the same toaster. Consciously or unconsciously we wish to keep at a distance from these attributes of the modern consumer society: we believe we gain in prestige if in some way we can express our individuality. The majority of people believe they can do this with their house. This appears to them to be their refuge, and here emotion breaks out forcibly. Here, one wishes to be and to remain an individualist Hence the rejection of standardization, typification in building.

These problems must not be taken lightly. Building must not be regarded merely as an engineering problem. It has a sociological side because it is not a matter of indifference to society or the State how the individual lives.

Building has a physiological side, because in his house every person wishes to have a feeling of bodily comfort. Building has a psychological side: people want to feel secure in their house. And not least, all things considered building has a cultural side, or expressed in a better way, a cultural obligation.

The conclusions for our buildings made using the standard unit system: they should not only be correctly designed and economically dimensioned; they should also be well laid out. This calls for the services of the most skilled architects. Unfortunately only few of the constructional systems at present on the market comply with these design requirements. Frequently these buildings have risen without the supervision of good architects. This is short-sighted. If it is to become really successful, the prefabricated house using the standard unit system must not only be more economical to erect than when using past conventional methods, but also from the design aspect, it must comply with the most stringent requirements. Otherwise it will merely be regarded as a synthetic method of building.

Now a few words concerning construction procedures:

In the previous conventional method of building the architect drew up the design; had it approved by the customer; submitted it to the authorities, prepared the specifications and then handed these to individual craftsmen and contractors. The architect also supervised the way in which the works were carried out on site, acting as the site supervisor.

Does this sequence of operations appear logical also for a standard unit system? My immediate answer must be that I consider it essential to have a change in the sequence of operations and a transfer of responsibility.

Before an architect can draw a design in accordance with a standard unit system for building, from some side or other such a standard unit system must have been developed. Usually this is done by builders having the necessary pioneering spirit who have pushed forward in this sphere and have studied a standard unit system. Naturally, close collaboration between architects and firms is desirable right from the initial plans, but in actual practice the architect usually lags behind.

If a building firm deals with all questions of prefabrication, mass production, standardization and industrialization, there is the following danger. If it is a firm of steel constructors, then naturally a steel building will always emerge. If the aluminium industry takes up these questions, there will always be aluminium houses, whilst obviously the timber industry constructs only timber houses. I would like to question whether this always provides the optimum solution and whether the correct material is always used at the right place.

In my opinion those firms which develop such standard unit systems are faced with excessive demands because of course they not only have to design the basic elements for the supporting structure and indicate the wide variety of their use, — but also have to undertake the entire installation and internal finishing. Usually one or other of these problems is given too little attention.

If we had a standard module, the structural steel firms could devote themselves solely to the supporting structure whilst other firms experienced in interior work could take over the task of developing suitable components for the internal structure and of marketing them. Then however, the question would arise of who was to co-ordinate all the works on site. The architect is not in a position to do this, because special problems appertaining to preparation of work on site, the use of mechanical handling equipment, plus suitable programming have to be solved which are outside the knowledge and experience of the architect.

Concerning these problems I personally have had good results with the general contractor. He takes over and is responsible for the completion of the building, on a turn-key basis, at a fixed price and at a certain date, consequently relieving the architect of all organizational problems. However, the general contractor can also relieve manufacturing firms of some of their difficulties. The individual specialist firms then merely have to worry about their own specialized problems and the general contractor has the task of overall co-ordination.

It is probable that many problems which are related to the standard unit system can be solved more rapidly and efficiently than in the past, by employing this demarcation for the sequence of operations and, as a result, the clear limits of responsibility. In this connection I am thinking of the question: how large must the number of items in a series be so as to achieve the optimum economic effect?

The number of items would, in the case of a component of the supporting structure, be different from that for a component of the internal structure. If, however, both investigations are left to the same firm, the results can easily influence each other. More unequivocal results are obtained if these are examined by separate firms. The general contractor can then make use of the optimum results from each individual firm and integrate them on his own responsibility to a complete whole.

Perhaps my thoughts are ranging too wide outside the compass of this Working Party. Even so, I am glad that I have expressed them and it would be important to me in the subsequent discussion to hear perhaps the opinion of the experts among you concerning the key concept "general contractor".

Commentary concerning the photographs

Photograph 1 shows a kindergarten in Holland. This, however, is not built of steel, but of reinforced concrete. Perhaps I ought to apologize for showing you this example, but it illustrates clearly and unequivocally what we mean with a standard unit system (a "closed" system). It concerns one small element repeating itself in many variations, and one large element harmoniously included.

The basic elements do not need to be square or rectangled, but instead, as shown in photograph 2 (Building at the Italian Olivetti Works in Sào Paulo) can consist of triangles or hexagons. It is important only that these basic elements can be positively interlocked, as can be seen especially well from the plan (Fig. 1). This example shows the manifold nature of this type of closed system, of which one certainly cannot say that it is boring or monotonous. Unfortunately, this example is again a reinforced concrete structure.

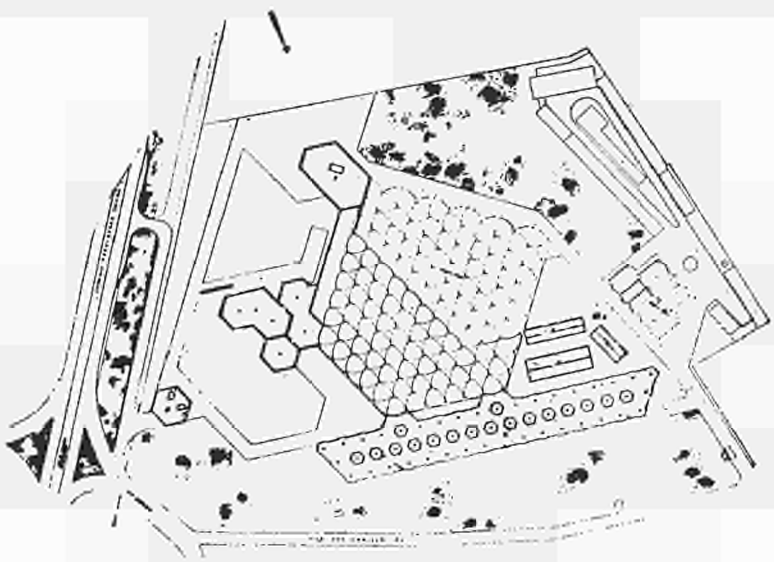


Fig. 1

Photograph 3 however shows a steel structure, a supermarket in Barcelona. Here we find individual square elements which are uniformly placed side by side. Personally, I the find preceding two examples of greater interest because they show better the many possibilities of the standard unit system.

In photographs 4, 5 and 6 the method of construction which I described as the "open" system are illustrated. These are school buildings in accordance with the English Brockhouse system. Here the individual components have been standardized—columns, cross-beams, floor beams—and in combining them a wide variety of structures can be produced, as can be seen from the photographs from different sites. All individual components are prefabricated and are very well designed. The entire structure, however, shows clearly that everything has to be built up from a common base, because otherwise of course standardization of the individual components would not be possible, and it would be difficult to interlink them. With this open system the question of architectural layout plays quite a decisive rôle.

It depends entirely on the architect as to how he masters the standard unit system to make full use of possible arrangements. Without doubt, photographs 7 and 8 of a primary school are very convincing. The school was erected in accordance with the previously mentioned English Brockhouse system. It proves there are no restrictions as regards layout in this system.

The next example (9) is not quite so convincing. The arrangement of the gable, and the way the staircase fits in with the longitudinal façade, shows in many ways the limitations which can be encountered when using the standard unit system.

An open system necessitates that all the numerous individual components be matched well to each other. Once again a school built with standard units, by an Italian firm. With this open system, importance attaches above all to the matching of the finishing and fitting-up work and the skeleton structure. It does not suffice that the skeleton alone is clearly designed and well constructed, as can be seen from photograph 10; the finishing and fitting-up work must also be equally convincing and must not be too greatly influenced by the framework.

In the picture of a hall (11), it can be seen that, even inside the building the dominating impression is one of standard units.

Take an example from France, once more a school (12). One cannot very well say that this concourse greatly tempts the user to walk up and down here. In my opinion, in this standard unit system the supporting structure has been made the focal point of all considerations and too little attention has been devoted to the finishing and fitting-up work. The interior of the classroom (13) also leaves much to be desired from the design point of view. The heavy beams no doubt interfere with the scale of the room.

I am not presenting these photos in an endeavour to criticise, but to show—and from my own experience—how difficult it is to cope with this method of building, which is so heavily orientated from the engineering viewpoint.

This problem has to be tackled again and again, with great perseverance, patience, tenacity and honest effort. No success is achieved if one of the participants believes that he has solved his task—possibly at the expense of others—but all participants must co-operate and seek the best solution; the solution which complies with all requirements.

To show one example of industrial building, I refer to photographs 14 and 15 of a warehouse shed which I erected using the standard unit system.

As can be seen from the internal view, the supporting frame work is based on a clear grid system. Such a warehouse may be extended in any direction and to any desired dimension, which merely has to be a multiple of the basic grid.

However, the most important feature of this warehouse shed is not so much the supporting structure as the arrangement of the external walls and the roof. Both parts, wall and roof, are similarly made completely from steel. The roof (16 and 17) consists of individual sheet-metal units. (dimensions 1 $\times$ 5 m.) These units are clad with plastic on both sides so that they need neither painting nor any other maintenance. To stabilise them the edges are flanged on all sides and the actual surfaces are provided with beads. The sheet-metal units are butted to each other and their joints are sealed with a plastic adhesive. Instead of the sheet-metal panels, attachments for roof lights can be inserted. Depending on the desired light intensity inside the building, these roof lights can be spaced at larger or smaller intervals within the roof area. Photograph 18 shows the view looking on to the finished roof. The walls consist similarly of plastic-clad sheet-metal units, which are profiled to provide stability and which are placed one beside the other in strips (19). In the case of this structure we can really speak of absolute agreement between the framework and the finishing and fitting-up work. Here we have one of the rare cases where the entire building consists almost completely and solely of steel and consequently everything is undertaken by one firm. In my opinion, a logical and consistent solution like this is hardly to be achieved in the building of houses, only in industrial building.

The far-reaching consequences of a logically used standard unit system, are shown in photographs 20 and 21 of the new German University Building at Marburg which, based on detailed investigations, will be erected entirely in accordance with a standard unit system. (Fig. 2)

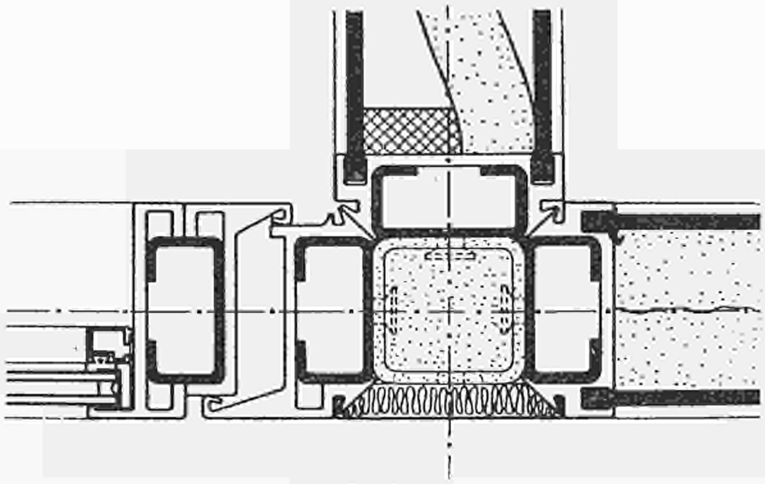


Fig. 2

The supporting structure consists largely of reinforced concrete. However, largely prefabricated steel components are employed for the finishing work.

It proves that in the standard unit system different constructional materials can be quite easily combined.

I do not wish to demonstrate technical matters to you however in connection with the example of Marburg University, but instead the effect of the standard unit system on town building. The site plan of this new University quarter (22) shows quite clearly that this overall conception is based on a standard unit system which is built up from a standard module; otherwise photograph 23 of a model would merely be an architectural trifle. Actually it is the result of very scientific investigation and represents the exhaustion of all achitectural possibilities which can form part of such a logical system.

I tried to prove that building does not lead naturally to a standard unit system, but instead tends naturally to deviate from this.

Photographs 24 and 25 of a single-family house (at Maison-Laffite) confirm this.

Figure 3 shows the plan of a single family house. This is not merely a design, but an existing structure.

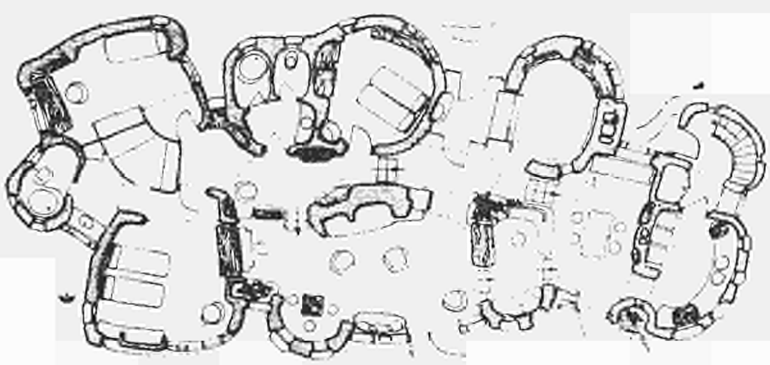


Fig. 3

Do not underestimate the desire of the individual who in the present-day mass society wishes to express his individuality. For my part, I can only understand this building as arising from this reaction, because, of course, it is not possible to say that the forms are very adaptable or capable of being regarded as the expression of an aesthetic attitude.

In fact we can almost state that here the attempt is being made to keep technique out at any price. The more rustic everything appears, the better. In passing it should be mentioned as a curiosity that the cornice consists of prefabricated concrete components.

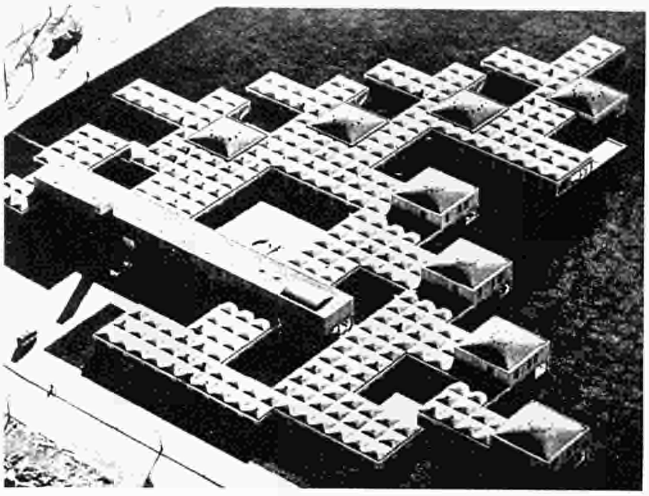
The next photograph (26), which illustrates a Bank building in Buenos Aires, cannot be regarded exactly as an expression of rationalization in building.

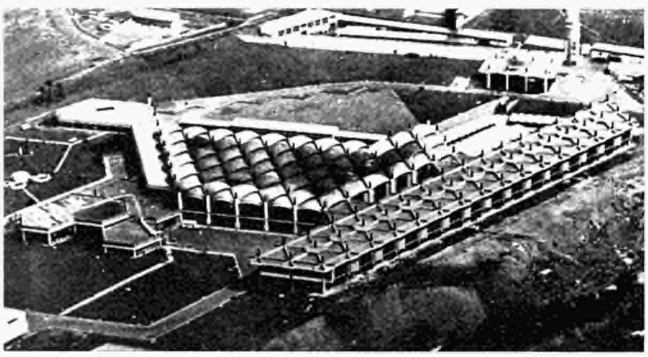
Probably a large portion of the structure could consist of prefabricated elements, but whether these components could be manufactured economically is another matter.

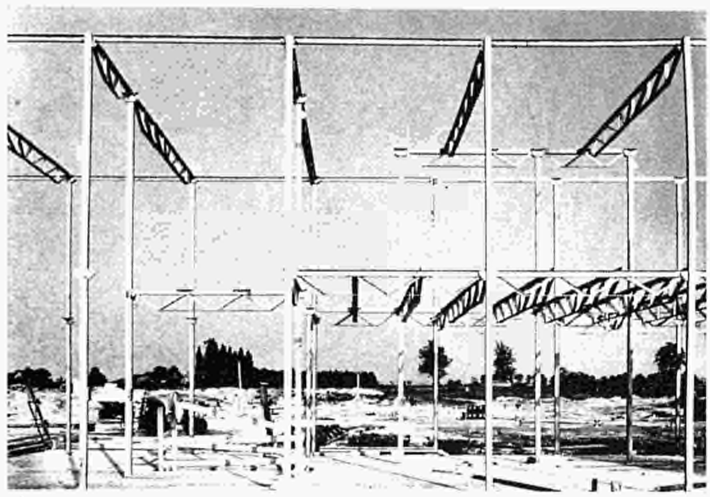
But even where economic considerations lead to prefabrication—as occurs with these houses erected in France and using prefabricated concrete elements (27)—, an attempt is made to counteract the engineering severity by irregular insertions consisting of natural stone.

In my opinion this is an attempt doomed to failure from the outset. However, we must recognize the secret longings expressed by this attempt and must regard the prefabricated house using the standard unit system, not merely as an engineering problem, but instead as a house for people who can live, not solely with the aid of engineering, but who in their buildings wish to proceed beyond purely engineering things.

I have intentionally shown a few extreme examples. You may say that these are exaggerations; this may well be true but exaggeration shows things clearly and it was my objective to demonstrate more clearly to you this so difficult sphere of "Prefabricated standard buildings and mass production of standard building units."







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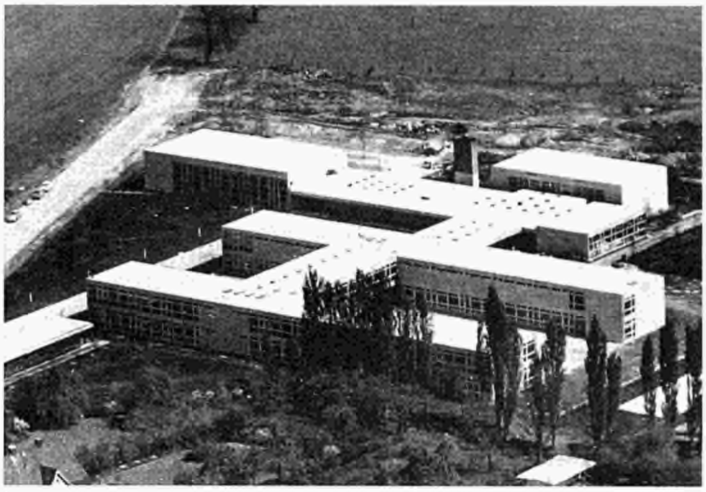

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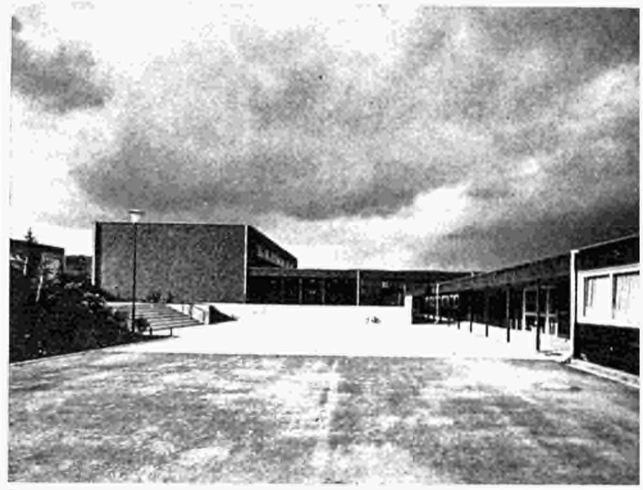

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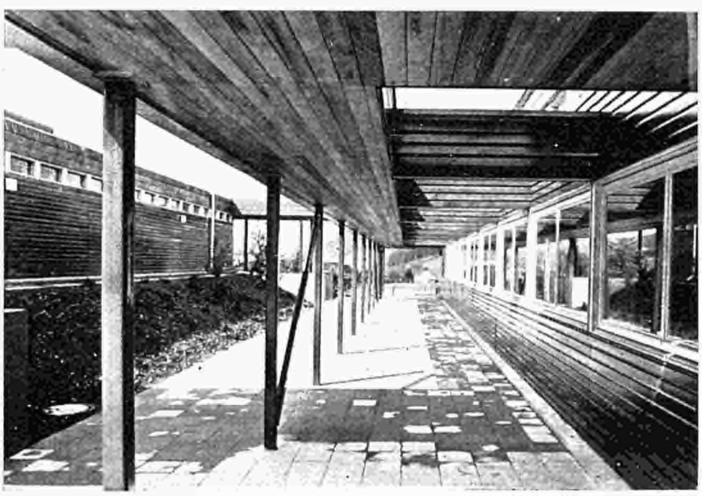

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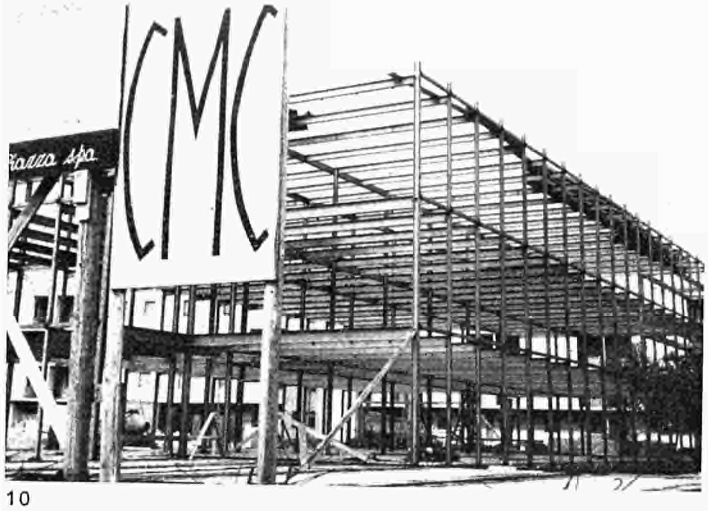

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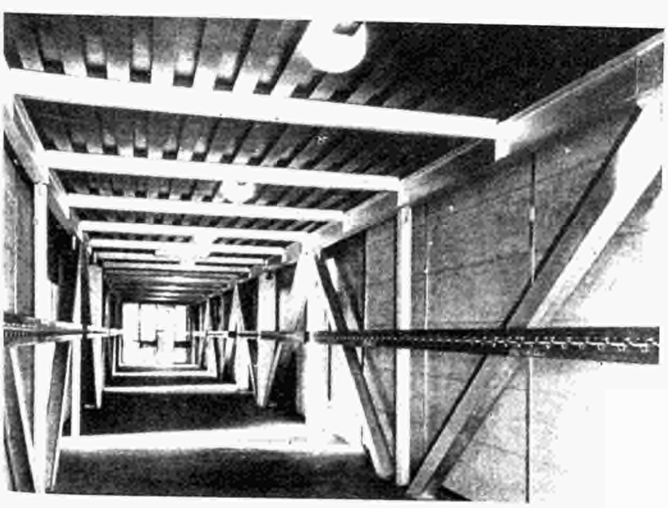

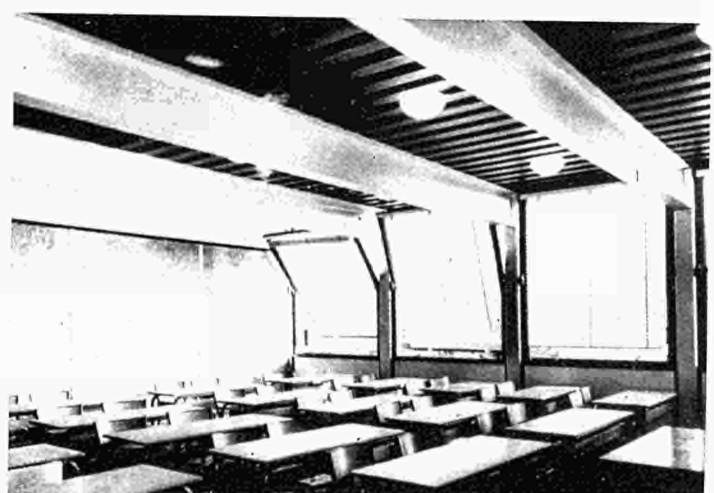

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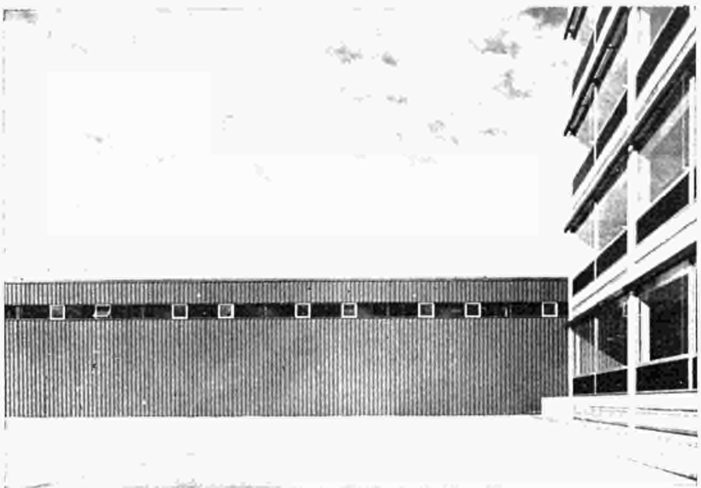
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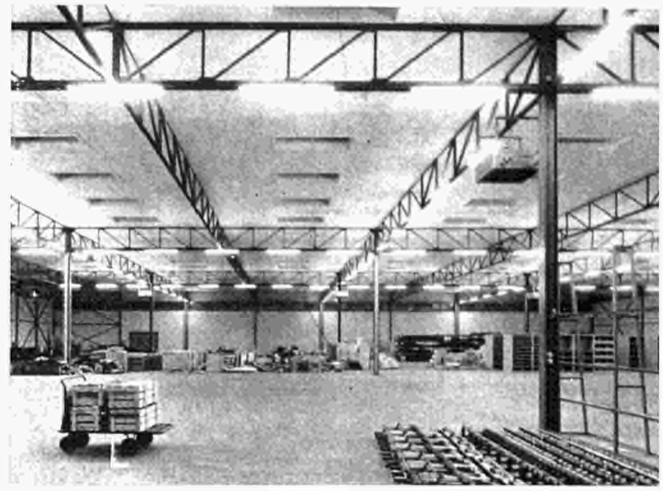
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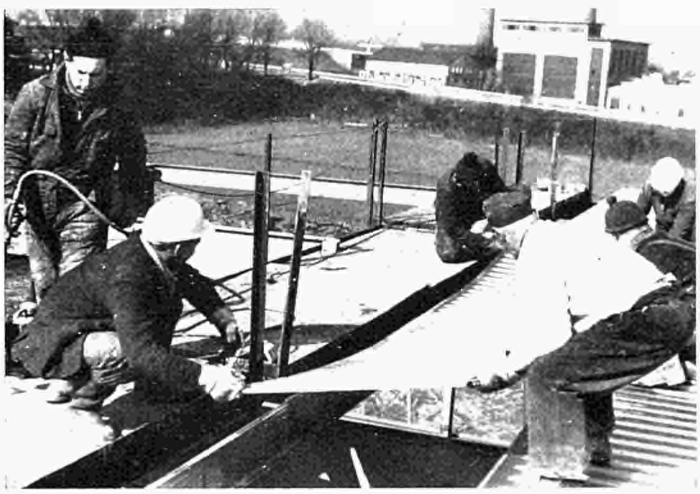
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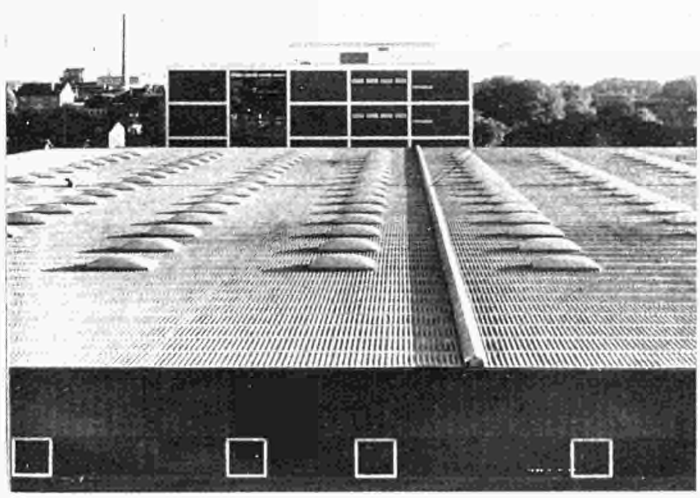
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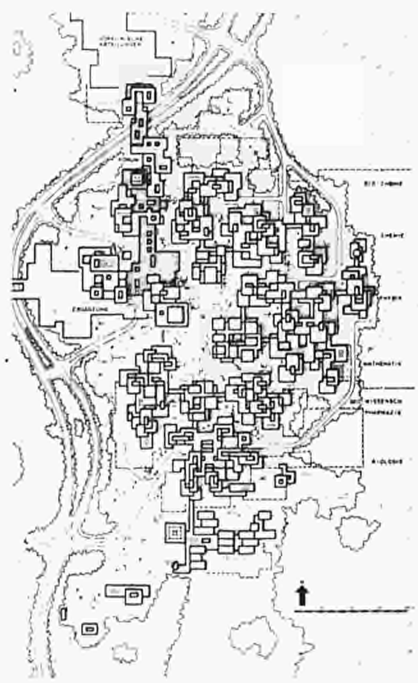

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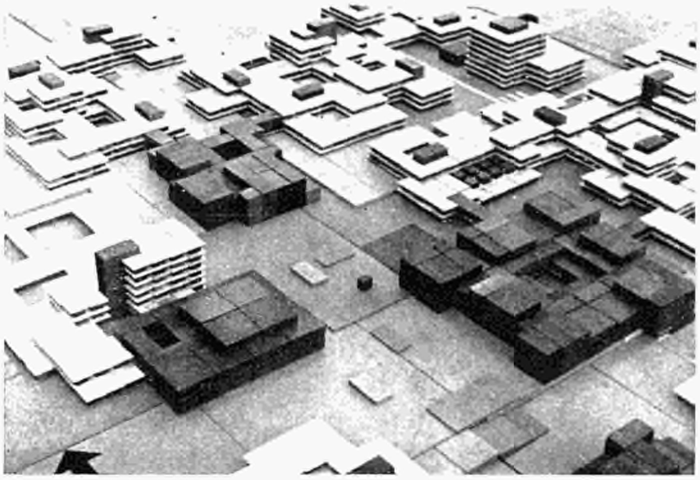

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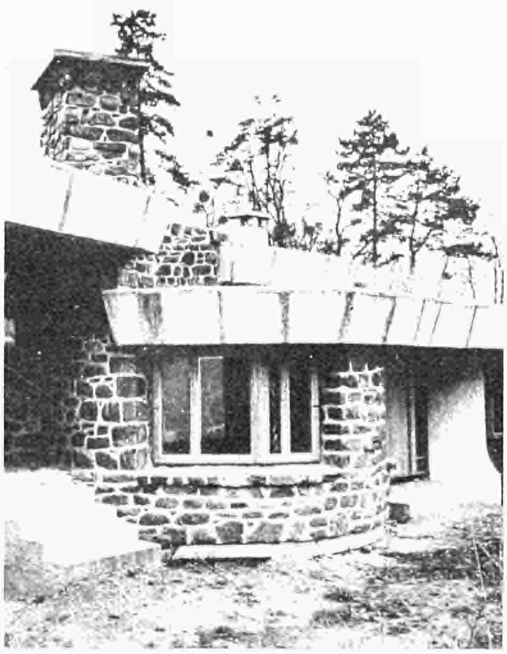

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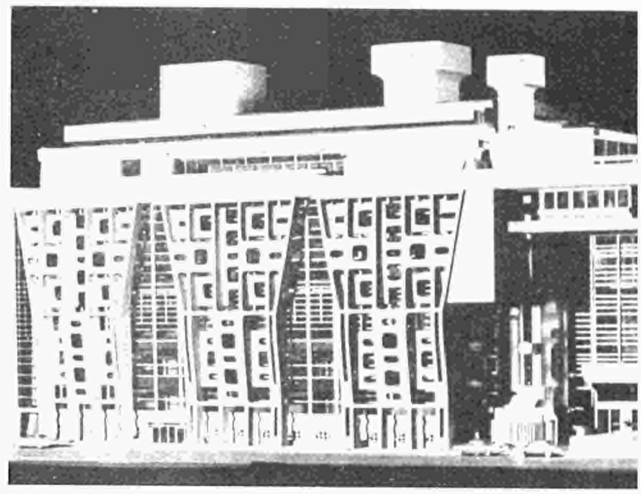

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27

Gabriel SCIMEMI

Criteria for the Standardization of Dimensions

(Original text: Italian)

Professor Sittig has given us an extremely interesting paper which should stimulate further study and research, aimed at broadening the concepts and conclusions he has presented to us.

In the first part of his paper, Professor Sittig deals with the standardization of the sizes of structural components in the special context of roofing for industrial buildings.

Today's audience also includes a large number of architects who are concerned with housing design, and it is therefore natural to ask whether the criteria mentioned can be applied to standardization in this field as well.

The laudable objective pursued in Professor Sittig's paper may be summed up as the relative minimization of *total detrimental variables*, an inevitable result of standardizing dimensions in component manufacture (the precise significance of this expression will be clear to those who have followed the paper), bearing in mind that production must aim at covering the entire range of requirements adequately whilst not supplying too large a number of varying types.

The range of requirements for factory roofing is conveniently expressed by the hump-shaped diagram plotted by Professor Sittig, on the basis of information gathered by Professor Neufert on a large number of cases.

In the housing industry, however, the range of requirements is of a completely different pattern. Basically, all the dimensions employed in residential design are based on one dimension only, that of the human being.

The dimensions of human beings, and hence their derivatives, obey certain well-known biometrical laws and can be conveniently expressed by a bell-shaped curve, the so-called "binomial curve", as demonstrated by Qetelet, which is in many respects very similar to the Sittig-Neufert curve (Fig 1).

Thus, at first sight, it would seem logical to approach the question of standardizing dimensions in residential buildings by analyzing the binomial curve in the same way as for industrial buildings using the Sittig curve.

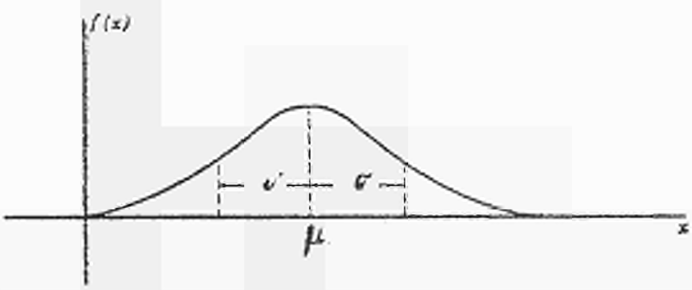


Fig. 1

From: Encyclopaedia Britannica article, entitled: "Probability and Statistical Theory".

Closer inspection, however, shows that this is inadvisable for two reasons:

1. A building, or even a room, nearly always contains human beings of varying sizes representing the entire range of dimensions rather than a single point.

2. The difference between the minimum and maximum possible dimensions is far more limited than in the case of industrial premises. We can exclude the existence of men whose height is perhaps two, three or even ten times that of their fellows.

   The bell-shaped curve based on physiological statistics is far narrower and more pointed than that based on industrial data; hence the integral of the detrimental variables is always fairly small, even if all requirements are met by a single dimension. This dimension can correspond to what statisticians call the "mode", *i.e.* the dimension represented by the maximum number of individuals in the population under examination.

It is a well-known fact, however, that residential housing production, which cannot be satisfied with a single standardized dimension valid for all requirements, also possesses a range of dimensions which can be represented by a diagram similar to that drawn by Professor Sittig.

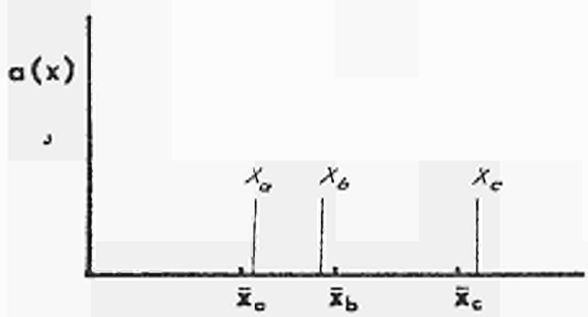


Fig. 2

The characteristics of this range however, are fairly diverse. It should be noted that the symbols x_a, x_b, x_c, x_d, etc. indicate "single" dimensions, each assignable to a different building component, and not various possible dimensions for a given component.

Thus x_a may denote the width of a staircase, x_b a floor thickness, x_c a window dimension, and x_d the height of a (living-accomodation) storey. As we have seen, each such component can be allotted a "normal" dimension, which automatically solves the problem of minimizing total detrimental variables.

As soon as the problem is solved, however, another important requirement arises in the residential (and non-residential) building industry, i.e., the suitability of the item as a "component".

To ensure that an item is suitable as a component the dimensions of the series must be multiples and sub-multiples of one dimension. The search for a series of dimensions sufficiently consistent with man's normal dimensions and strictly obedient to the component principle, necessarily entails modifications and thus reintroduces the question of minimizing detrimental variables as a basis for the selection of the most appropriate series.

There are doubtless many existing studies which postulate and solve this problem for us in various ways. The best known, and perhaps the earliest, is Le Corbusier's "Modulor". It is possible that he over-simplifies the problem by obviously thinking that dimensions derived from those of human beings can readily constitute a series (or rather two series) of multiples and sub-multiples without further modification.

It should be borne in mind that this theory is developed on the basis of what Le Corbusier calls "normal dimensions" according to his own personal concept. His system is based on a geometrical progression (approximate ratio 1.161 derived from the "golden section" of the classical architects (Fig. 3) whose first 12 terms add up to 226 centimetres (a man standing with his arm raised), plus a second absolutely analogous progression whose first 12 terms add up to only half of 226, *i.e.* 113 centimetres.

Whatever validity Le Corbusier's solution may have, there is no doubt that the subject is of the utmost theoretical and practical interest.

I regret I can offer no conclusions. A few minutes ago, we all heard Professor Sittig's extremely interesting speech, but despite the clarity of his exposition and the exceptional ability of the interpreters, this is not a subject which can be fully grasped in the short time that it takes to read a paper to this Congress.

I shall therefore confine myself to stating one or two ideas which came to mind while I was listening to Professor Sittig's outstanding account.

I am convinced that the system of minimizing total detrimental variables, as applied to the combination of the two basic criteria — reference to the human dimension and modulation of dimensions for use as component — may suggest worthwhile avenues of approach as a logical extension of the methods proposed by Professor Sittig to the housing industry.

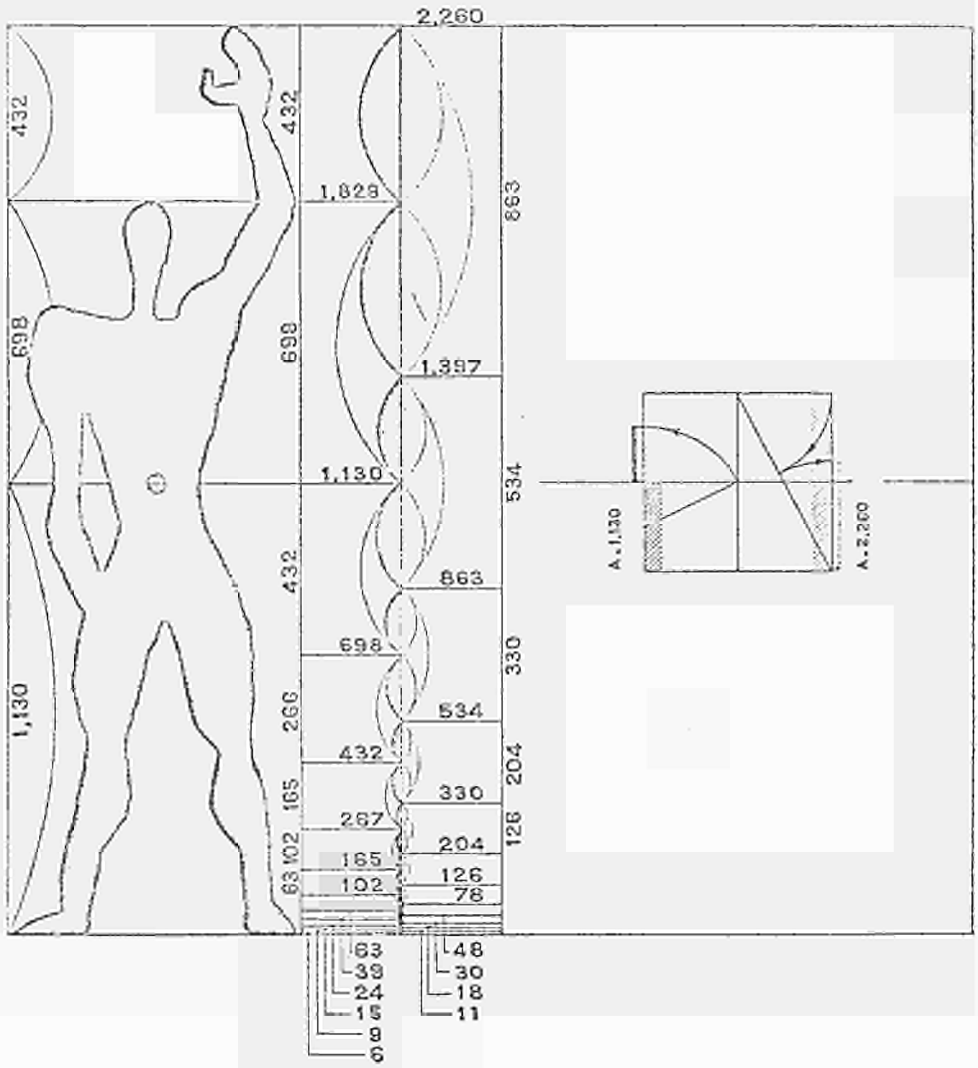


Fig 3

Modulor, a new human measure.

From: Le Corbusier, 1938-46, Editions Girschberger, Zurich, altered, until it suddenly disintegrates.

D. E. HAGEMAN

Mass-Production and Social Factors

(Original text: Dutch)

In addition to the two reasons given by Mr. Sittig for the lag in industrialization in the building trade, I feel it needs to be borne in mind that the normal building contractor tends primarily to be influenced by the past in his approach to production, which makes that approach very much an artisan one. It is doubtful therefore whether such contractors will ever come to think in terms of industrialization.

Quite apart from his mental approach, it is doubtful whether the average contractor is financially in a position to cope with the capital expenditure needed to ensure rational production. After all, as Mr. Sittig himself pointed out, mechanization plays a very important part in this, and mechanization is initially most costly.

To Mr. Sittig's observation that the industrial approach can surely be expected to be current among steel manufacturers, I would add that it is equally important for the resulting mass-produced article to be accepted by society.

An intensive drive will be needed to bring it home to people that the idea that "mass-production means lower quality" is wrong.

It is also necessary that the authorities should encourage the acceptance of mass production by instituting more sensible, practical rules and greater uniformity in their local regulations.

Georges REPECZKY

Professional Rules and Practices Versus Industrialization

(Original text: French)

The company I represent has been connected for many years with building work, in particular the construction of schools and housing for repatriated settlers from Algeria

I would like to describe the problems which these projects involved with regard to designing, to fabrication and to assembly and erection.

At the design stage, the industrialization of the building process is being held up by the promotors, who are not always prepared to accept the demands which this imposes, and by architects, who must to some extent educate themselves if there is to be mass production of serial components. Both need to be convinced that, in return for their acceptance of certain restrictions, industrialized building can offer them the possibility of economically viable designs that can quickly be effected.

It would be helpful if standards could be recast in order to establish modules in the three dimensions to serve as a basis for mass production of frames, curtain walls, floors, partitions and ceilings, staircases, and inside and outside joinery work. Housing units are at present measured in square metres of floor space and head room.

The fire regulations should be restudied and modified, for they tend, for no really good reason, to operate against the use of steel.

Each trade works to its own degree of precision. We will have to redefine what constitutes the main structure, owing to the greater precision required in industrialized building.

Attempts to combine industrialized with traditional methods are not always very satisfactory: delays are apt to occur in the operations owing to the workmen's unfamiliarity with the job and the difficulty of using traditional materials along with prefabricated components.

At the fabrication stage, special demands by promotors and architects seeking to break away from industrial standards interrupt the series and necessitate extra adjustment studies which bear heavily on costs.

At the assembly and erection stage, industrialized building demands adaptability from the contractors. With the exception of the foundation work, for which the firm's ordinary workmen will be employed, a more highly skilled labour force needs to be trained for the jobs that demand greater precision and more concentration on performing the work really well. Even the contractors' actual equipment will have to be altered to ensure the necessary precision in assembling and erecting the structural components. I understand that the French lifting-equipment manufacturers are now engaged in studying this question.

These various problems have been especially prominent in residential building; they have proved easier to tackle in the case of industrialized building for educational purposes. Here the authorities commissioning the project were able to oblige the architects to accept the modules and standards devised for industrialized building. Some exemptions were granted in respect of the usual rules, allowing the use of materials other than those traditionally employed, and the mass production of prefabricated components for ceilings and partitions.

This experience shows that teams can be formed for a combination of jobs — erection of main and subsidiary frameworks, and installation of curtain walls, roofs, staircases, linings and facings, ceilings and partitions. This made for easier co-ordination with the subsidiary trades involved in the erection.

Frank E. S. WEST

Prefabrication of Buildings in England

(Original text: English)

There are two points related to the prefabrication of buildings in England. The first concerning the fact that our major civil engineering firms have all got highly developed reinforced concrete and prestressed concrete departments.

Often these firms use all kinds of arguments to exclude the use of prefabricated steel on large projects. The only solution to this problem, which may only be related to work in the United Kingdom, is that steel fabricators should enter the general contracting field. This would appear logical in order to make the full use of steel, which gives a light-weight construction. Therefore steel fabrication can best be used if all the other units are of light weight (in filling and floor units, for example). On the other hand, it would be most necessary to use other materials where logic demands their use. Logic is a fine word to use but we are in England faced with this great difficulty that often steel prefabrication is ruled out by influential pressure rather than by logic.

The second point, (again this may only be related to British production, but from the two previous speakers I gather that this is not so) for truly mass production methods the dimensional tolerances of steel sections, with the exception of tubular, and cold form sections, are far too high. It is a matter of great importance to efficient fabrication that the sections used be to exact dimensions without twist lozenging or other distortion, and I feel very strongly that the reason why in England we have not been able to make full use of mass production methods is that our sections have not got the high degree of accuracy required, therefore, our fabricators don't think that precision is a terribly important thing. With tubular and cold form sections, of course, it is easier to fabricate accurately but in rolled sections this matter of dimensional tolerances is of primary importance for manufacturers today.

Pierre MESLAND

Remedies of the Disadvantages of the Use of Steel

(Original text: French)

In a very interesting introductory talk, Professor Jan Sittig showed that steel has also its disadvantages, among which he included corrosion, difficulty of heat and sound insulation, high cost and inadequate safety in case of fire.

Monsieur Jan Sittig knows quite well that there are very obvious remedies to these faults.

Corrosion

Steel, although one of the oldest known products, has remained a "shy" material: it "blushes" on the surface under the action of water or humidity. This is at least an external manifestation of "frankness" on its part, which is preferable to the "shy" behaviour of materials such as wood which can rot, be gnawed away by termites, have its inside completely removed without its external appearance being altered, until it suddenly disintegrates.

Our old Eiffel Tower is still standing quite happily after 75 years. It is repainted every seven years, but this is done mainly because above the second platform, which overlooks the Paris punch-bowl it is exposed to violent winds containing particles of dust which attack the paint in a way rather akin to sandblasting.

There are many examples of steel structures exposed to heavy weather: bridges, viaducts and pylons. These can be protected from corrosion by painting at regular intervals. This technique has been improved by surface preparation which descales the steel. It consists of burning with a blowlamp, brushing after allowing the scale to oxidise, sanding, hot or cold zinc spraying, etc. The welded construction technique and the concept of pre-assembly have eliminated riveted gusset plates with hollow spaces where rain and moisture could collect or filter into the joints. There are simple streamlined forms where corrosion cannot develop. The fine Grand Duchesse Charlotte caisson bridge, under construction only a few yards from here, is an excellent example of this.

Steel frames for houses on the other hand are protected from bad weather. Enclosed in the main walls, they run no risk. Where they are partly exposed, they are quite small, and it is easy to protect them at little cost by the methods mentioned above.

It is more important to protect thin steel parts. 0.1 mm of corrosion on the two surfaces of a 1 mm thick flat product reduces its cross-section by 20 %. With thicker, 4-10 mm and more, types, this amount of corrosion is negligible as regards strength, being 2 % for 10 mm, which is the average thickness of rolled-steel framework. With a safety coefficient of 3-4 this reduction in the cross-section is negligible even without maintenance.

Thus it is flat products which must be efficiently and durably protected. This is especially true of steel parts such as roofs, curtain walling, panelling, window-frames and fittings, cladding, string-courses etc. when the external surface is exposed to the elements.

The remedies:

Continuous galvanizing by the Sendzimir process which deposits 25 microns of zinc. This has nothing in common with the old-style dipped galvanizing of wash-boilers and washing-up bowls. Protection can be completed by appropriate painting, continuous lacquering or plastic-coating. A decorative element can also be obtained with a choice of colour and finish.

Hot-enamelled steel, invariable in protection and colour, is at present used for the exterior breast-panels of curtain walling and front-panneling.

The properties of chromium-nickel 18/8 and molybdenum 18-10-3 stainless steels are also well known.

Heat insulation

We should remember above all that steel in comparison with other metals has a low thermal-expansion coefficient:

11×10^{-6}, approx. 1 mm/m for 100° temperature change, as against 2.3 mm for aluminium (23×10^{-6}).

Let us compare the heat-conductibility coefficients of ordinary steel and of light alloys. These are in the ratio of 110 for mild steel to 320 for AG3 steel. This shows that light alloys are on average three times better as conductors than ordinary steel. Their tensile strength is on average (with the exception of duralumin) only half that of mild steel. That means that for the same strength their section must be twice that of steel. With a heat conductibility three times greater, they therefore offer a six times greater passage to heat and cold than steel.

Stainless and resistant steels have a conductibility coefficient of 39, *i.e.* one third that of mild steel, with a tensile strength twice as high (non-cold-worked steel). This 18/8 stainless steel has a conductibility 32 times less for the same strength than light alloy AG3. This is why two years ago, in Italy, I saw this great heat resistance of stainless steel used as a mechanical link and heat insulator in the form of a 4 mm thick strip between the external and internal breast-panels of the curtain walling. This solid mechanical link acts like a heat cut-off. When the breast-panels are of steel to comply with the French regulations (which prescribe a heat transfer coefficient where K is less than or equal to 1), there are many ways of ensuring heat cut-offs which have been approved by the C.S.T.B.

Soundproofing

Of course a sheet of steel resounds like a drum, but in the construction of a motor car, for example, it is sufficient to crush a few balls of a special product on the internal surfaces to sound-proof the doors and bodywork.

Movable steel partitions are made with painted or plastic-coated steel surfaces which reduce noise by 25-40 decibels. This is far better than with the traditional partition wall. Weight is of course an important noise-reducing factor, but it is also possible to "break down" the noise by combining several materials with different frequency responses thus forming a screen against the spread of sound.

Counter-floors in perforated steel with an air cushion and an absorbent layer of mineral wool have an absorption coefficient of up to 80 %.

Fire

Of course, steel gradually loses its strength at temperatures from 550 °C upwards. It can be protected, and the regulations provide for or even demand this. The regulations especially in France, are rather too severe having regard to technical progress and to fire-fighting facilities. Some regulations are such that the protection is of itself likely to increase the demands on the steel. They are especially hard on steel and are about to be relaxed. And let us remember that the human body cannot stand such temperatures and that, above all, there must be rapid means of escape, leaving the firemen to their task.

In conclusion, steel remains a young material, at the forefront of progress by virtue of the constant improvements in its mechanical properties, its protection and its appearance. Steel has been tried and tested, whereas certain new materials may prove disappointing despite their early promise.

Anne VOLBEDA

Tolerances and Fits in Building Construction

(Original text: Dutch)

The introductory speaker has considered industrialization of building under four headings, namely: standardization, limitation of the range of types, prefabrication and assembly.

In regard to the first two stages he has mentioned planning and has given an example of this in his deliberations. As regards the third and fourth stages he has proposed, respectively, solution of the problem of tolerances and fits, and the organization of standardized assembly without the need for adjustments to be made to the components. I should like to discuss these points in a little more detail. I do so against the background of many years of fruitful co-operation with Mr. Sittig.

Industrialized building construction here means the prefabrication of standardized components and their assembly on the building site without the need for adjustments.

The process begins with the manufacture of the components, which are made in many different factories. Not all the components produced reach the building site. Some of them may be rejected following inspection at the factory.

There eventually arrive at the building site some components with a certain degree of inaccuracy, dependant on the method of manufacture and the system of factory inspection. The assembly of these components on the site produces a joint in the end-product, the inaccuracy of which is deter-

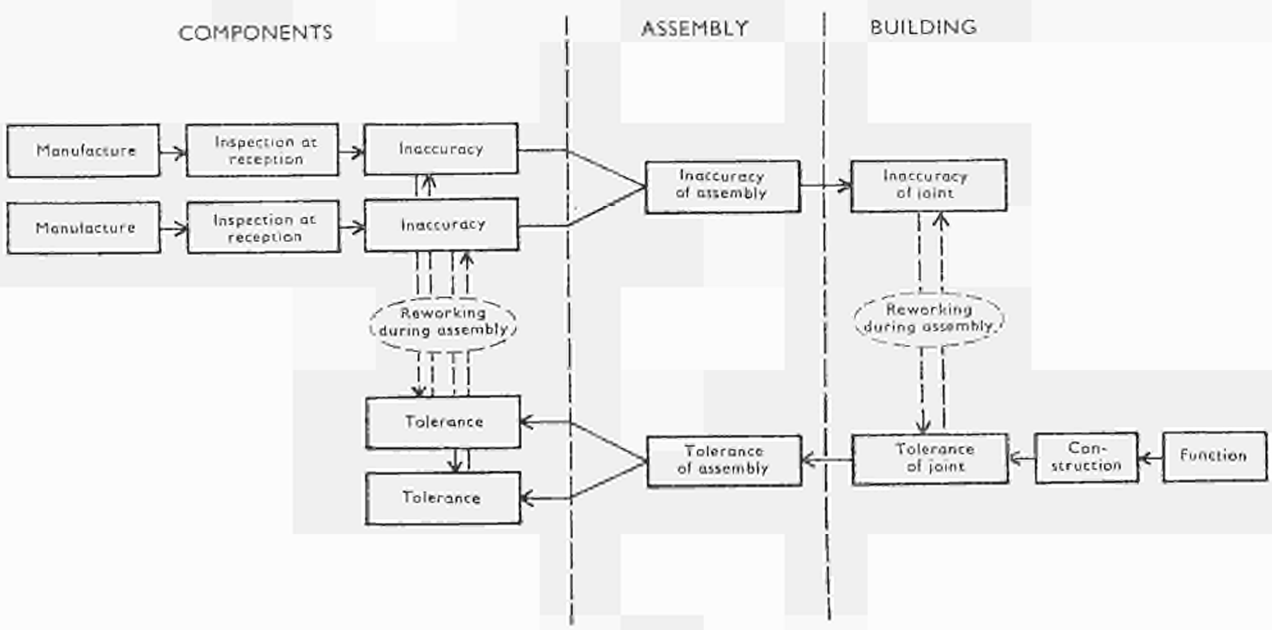


Fig. 1

mined by the inaccuracy of the individual components together with that of assembly. This process is depicted synthetically in the top line of the diagram (Fig. 1). This line depicts the actual situation (what can happen in practice) and must be read from left to right.

The limits of permissible inaccuracy of this joint are defined by functional, technical and, sometimes, aesthetic requirements; to put it more simply, the joint may be neither too wide nor too narrow.

The permissible inaccuracy of the joint is called its tolerance. This tolerance may be derived from the function to be performed by the assembled components and from the constructional solution adopted by the designer to achieve this functional efficiency. If two steel gable beams are laid on a steel upright the joint tolerances will be different according to whether they are welded or connected by a bolted joint. The total joint tolerances may thus be sub-divided into two parts:

(a) Component tolerance, and

(b) Assembly tolerance.

The former can be presented mathematically in many ways. A satisfactory situation exists when there is a reasonably close agreement between the inaccuracy and the tolerance of the individual components, of the assembly and of the end-product.

In contrast to the top line, the bottom line of the diagram is thus analytical in structure. This line, which must be read from right to left, states the normative approximation (what should happen).

In a balanced approach to the problem both lines of the diagram must be considered. An example of this approach is furnished by the European Coal and Steel Community's Experimental Building Scheme II in which the problem of fit has been tackled in the manner described. In practice, however, this course is not often followed. Cases are common in which regard is paid only to the top line of the diagram, i.e. in which there is a tendency to accept the inaccuracy of the components as delivered, (those resulting from the method of manufacture and the system of inspection), and to determine the tolerance on the basis of the accepted inaccuracies and not on that of the function to be performed. It will be clear from the foregoing that this approximation, in which "what can happen in practice" is dominant and "what should happen in theory" is completely subordinated, is a very one-sided approach.

I should like very briefly to describe how this problem was solved, not merely theoretically but what is even more important, practically, in the second E.C.S.C. Scheme, in connection with the fitting of wooden doors into steel doorframes.

The starting point was the usual jointed construction in which the door fits into the rabbet of the frame. It is commonly found that the doors are too big and need to be made to fit by sawing and planing on the site. In industrialized building this need for reworking during the assembly stage must be avoided. It must be possible for the prefabricated, mass-produced door to be assembled without adjustment and to provide a proper fit. For simplicity's sake we shall confine ourselves in the present example to the horizontal dimensions of the door and frame. Measurements taken in practice have shown that the joint may not be narrower than 0.5 mm or wider than 4 mm. This gives a joint tolerance T_V of 3.5 mm. From this joint tolerance are derived the door, frame, and assembly tolerances, by means of the formula

$$T_V^2 = 1/4\, T_D^2 + T_M^2 + T_K^2$$

in which:

T_V = joint tolerance

T_D = door tolerance

T_K = frame tolerance

T_M = assembly tolerance.

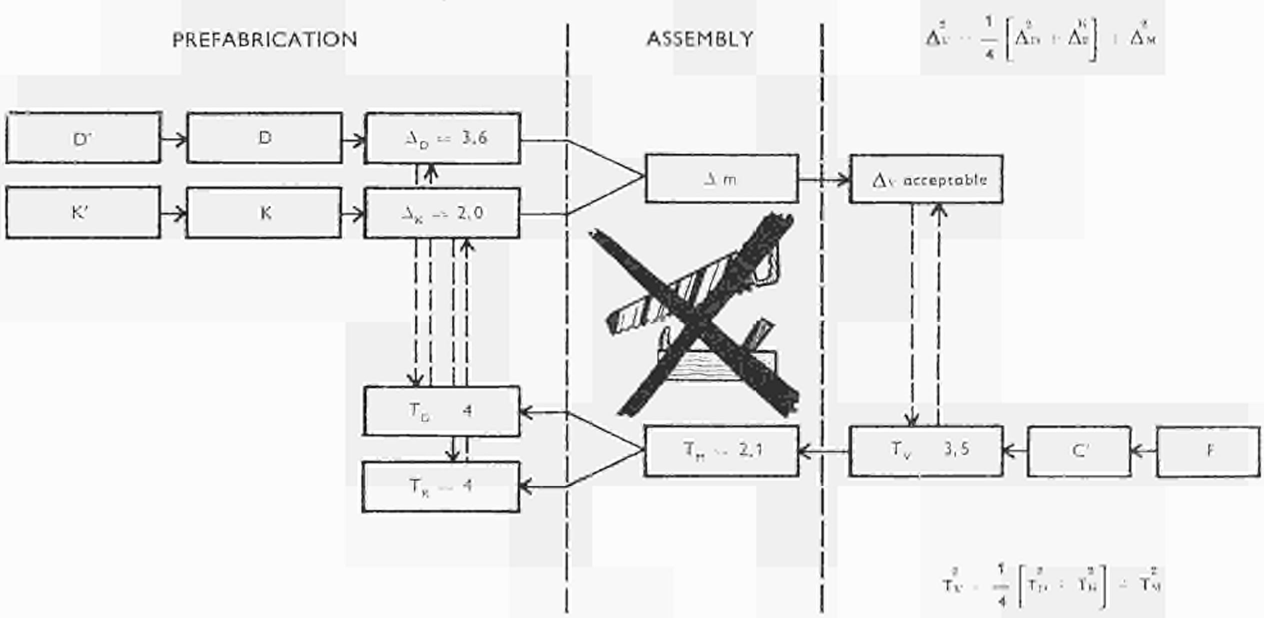


Fig. 2

On the basis of probability theory considerations it appears that the tolerances are (fortunately) quadratically and not, as is often thought in practice, linearly related. A door-measurement tolerance of 4 mm, a frame-measurement tolerance of 4 mm and an assembly tolerance of 2 mm thus together give a joint tolerance of 3.5 mm. The doors and frames are therefore ordered with a tolerance of 4 mm. Let us see how this works out in practice. The following are the results for the building site in Milan. The inaccuracy of the wooden doors as delivered to the site was $\Delta_D = 3.6$ mm and that of the steel frames $\Delta_K = 2.0$ mm. Since the inaccuracies bear a similar relationship to each other as tolerances, it will come as no surprise to learn that the fit of the standardized, mass-produced doors, assembled without adjustment, was judged to be good, *i.e.* $\Delta_V < T_V = 3.5$ mm.

On some building sites this result was not achieved (Fig. 2). The last diagram explains why. It shows the steel frame and the wooden door. The tolerance ranges for the door and frame are shown hatched. The inaccuracies are given for three building sites, on the top line for the frames and on the bottom line for the doors. The mean inaccuracy is indicated by a vertical line and the zone of dispersion by a white block. It is seen that on the Dutch building site at Heemskerk the degree of accuracy of the frames as delivered was satisfactory. The doors showed slight mutual differences but for safety's sake were supplied 2.5 cm too wide.

On the Gladbeck site, the door inaccuracies should have fallen between 33 and 37 mm from the zero line $T_d = 4$ mm; in fact, however, they lay between 30 and 38 mm ($\Delta_d = 8$ mm). The frames also deviated from the stated norm. On the Italian building site, both the doors and the frames fulfilled the requirements excellently. It is interesting to note that the quality of the assemblies after practical adjustments was judged to be considerably poorer than that of the assemblies which required no such adjustment. The following conclusions may be drawn from this specific practical example:

— the fit tolerance is the square root of the sum of the squares of the tolerances of the structural components;

— the manufacturers are technically capable of producing components within the prescribed tolerance range;

— it is both theoretically and practically possible to achieve a good fit of doors and frames by assembly without adjustments of prefabricated mass-produced components;

— the assembly without adjustment of prefabricated doors and frames results in a saving in man-hours; more significant, however, is the fact that this makes possible new industrial methods for the manufacture of pre-finished products;

— the example selected for the solution of the problem is of such a general nature that it can also serve as a model for other problems of fit.

Roger MORA

A Practical Example of Achievement : The Faculty of Letters, Paris (Extension to the Halle aux Cuirs)

(Original text: French)

Previous speakers have emphasised that to promote the use of steel there is a need for builders to have an all-trades planning office. Our company, which has always specialized in constructional steelwork, has had a general contracting department with an all-trades planning office for five years, and now produces complete steel-framed buildings ready for occupation.

We have for many years developed various methods and principles, the basic features of which can be seen in the completion of the Faculty of Letters.

This morning there was much discussion regarding corrosion of steel, and M. Mesland has clearly shown what and what not to do.

In this system used in the Halle aux Cuirs, the columns, of HE sections, are external, and thus are fully exposed to the atmosphere. These are not in thin-gauge 8/10 sheet, but thick-gauge plate which presents no great corrosion risk.

The question of tolerance also arose this morning. It is clear that the study of tolerance is the key to successful prefabrication, and in fact there are ideas which permit these problems (especially those of rolling tolerances) to be eliminated.

The design of the Halle aux Cuirs — steel framework with front-panelling — is original inasmuch as all rolling tolerances are relegated to the outside while the flanges of the internal columns are precision-aligned by means of a theodolite (see Fig. 1).

Moreover, the floor levels were fixed without need for adjustment on the site by supplying prefabricated columns with brackets welded on in the shop according to a model. This procedure had the following original features:

(1) greatest possible independence of all trades;
(2) ability to position the service-mains ducts where desired without altering the framework and floors (see Fig. 2);
(3) possibility of inserting service piping and wiring in the ducts at the same time as laying the floors;
(4) laying the floors on pantiles of galvanised ribbed steel sheet, starting from the top so as to make room for other work teams as quickly as possible (five floors);
(5) use of linear planning (see Fig. 3).

Results:

Beginning of planning	15/2/64
First foundation pit	24/3/64
First portal erected	22/5/64
Last square metre of floor laid	10/8/64 (12,000 m²)
Enrolment of students	26/10/64
First lectures	2/11/64

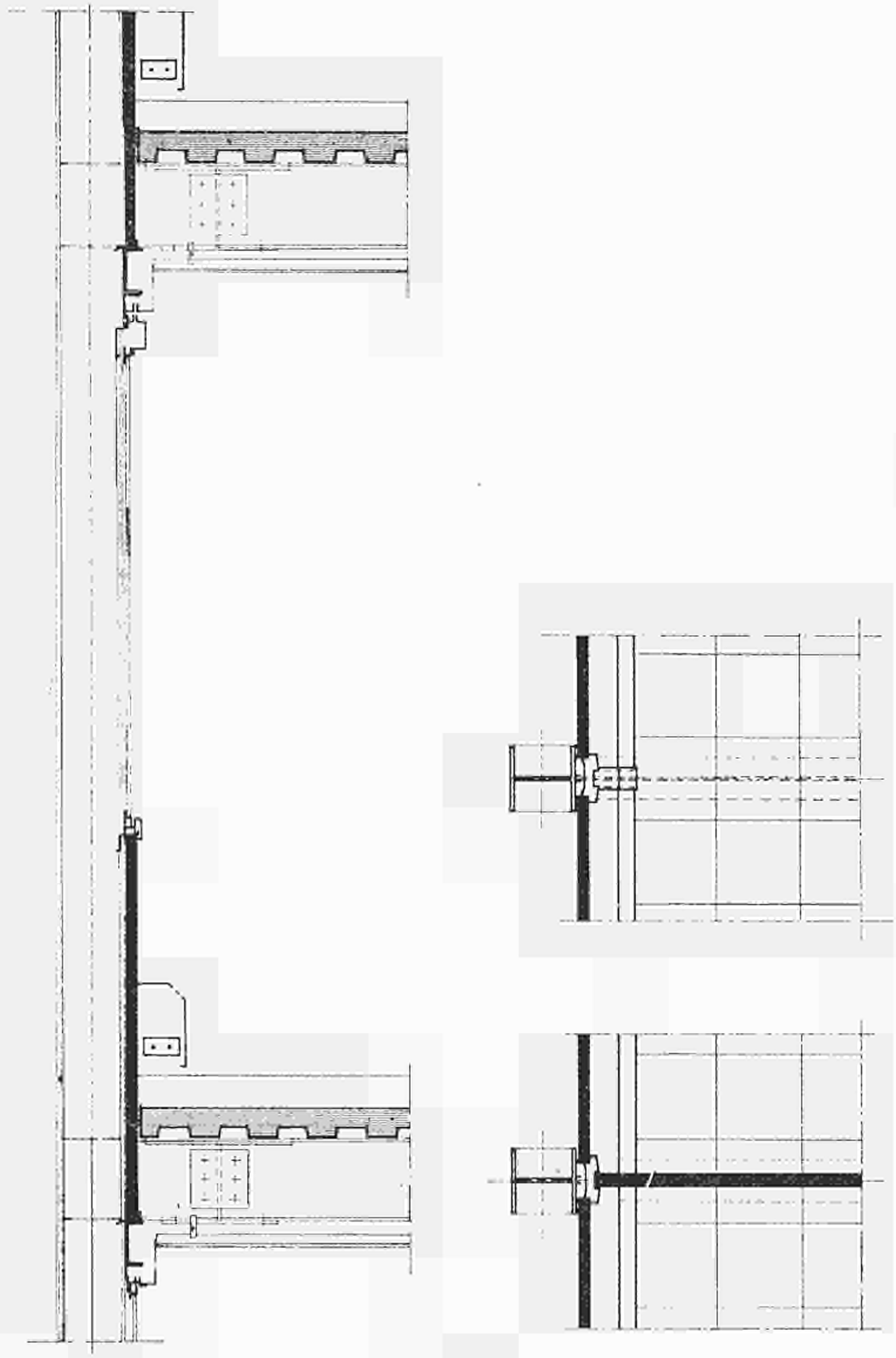


Fig. 1.

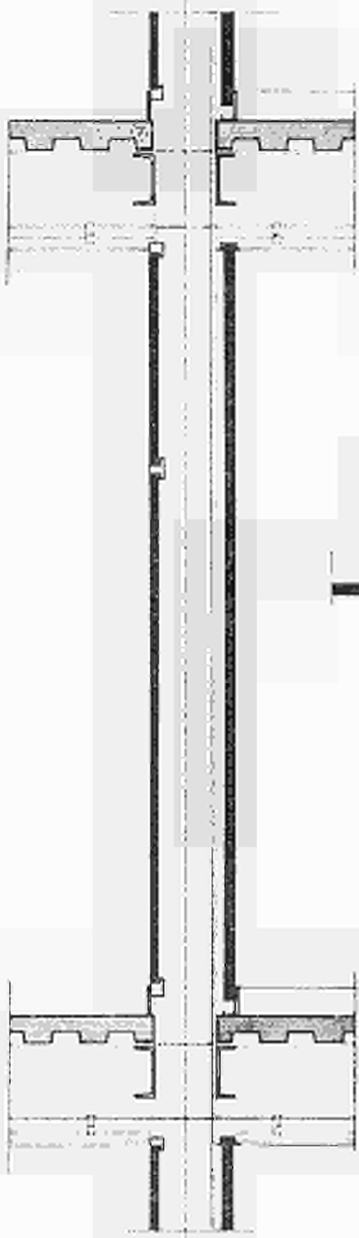


Fig. 2

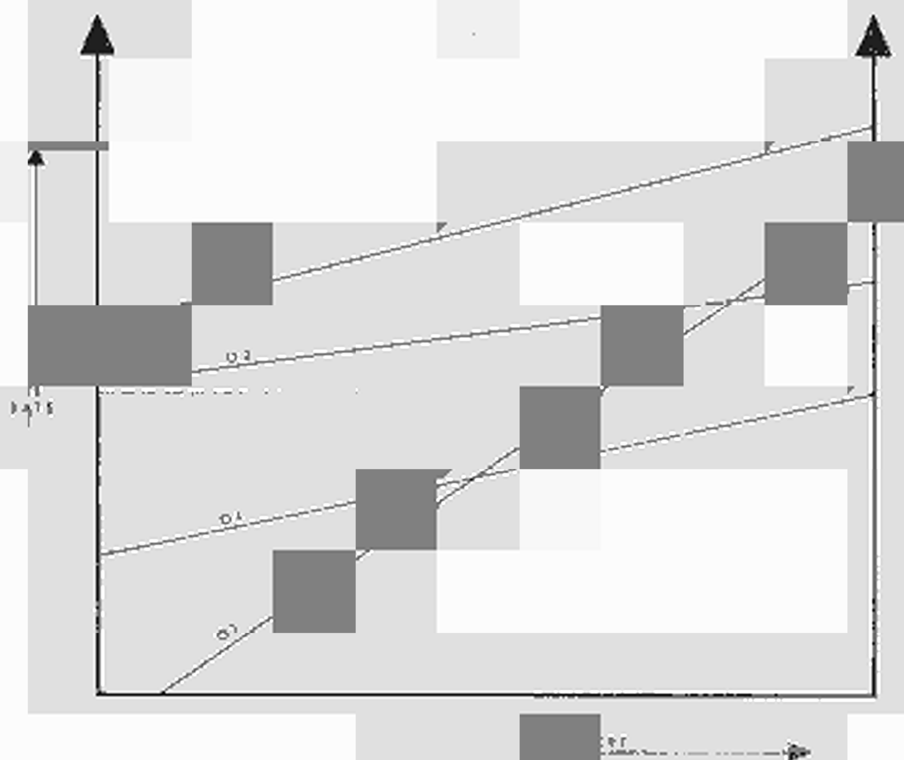


Fig. 3

Alexis ARON

Prospects for Much Greater Use of Steel in Building

(Original text: French)

In organizing this Congress, the High Authority has taken as its theme the maximum possible use of steel in building. This aim is not new: it has been in evidence since the creation in Europe, of the various offices for the propagation of steel. In the early years the problem was approached from a narrow angle, that of choosing the type of framework, metal or concrete. This was inevitable; for many years the heavy section (joist, U-section, angle-iron) remained the steel product most widely offered to builders. The initial concept of the all-steel house did not progress beyond the small one-family house, mainly for export to the developing tropical countries.

The field of steel utilization began to expand with the development of steel door and window frames and fittings; the increase was rapid but the tonnages concerned remained comparatively small.

The situation was, however, completely altered by the extraordinary increase in the production of flats. The curtain-walling solution, the success of which is well known, has resulted in steel sheet now being used on a large scale, in its various forms of painted, stainless, galvanized, enamelled, plastic-coated sheet, etc.

A further development, introduced some years ago, was the extensive use of steel for the other parts of buildings: external walls, roofs, ceilings, floors, stairs, internal partition walls. This development has great prospects but poses two questions. How is this new emphasis justified? In what form can the necessary supply of steel products be conceived? It is with these questions that I am concerned in this paper; I shall confine myself to the steel side of the problem.

The first question, justification of the new concept, can be sized up immediately from a simple statement of the problem. Let us take two steel-framed buildings: the first comprises, apart from the framework proper, only small quantities of steel; the second, on the other hand, is planned from the point of view of a wider use of steel throughout the building. Without going into figures, limiting myself to data from complete projects, I can say that the amount of steel used in the latter will be anything up to double that used in the former; indeed, this estimate is a conservative one, since some new buildings are using three times as much. With current programmes, and those envisaged for the near future, the size of the increase as the new concept is adopted, can easily be imagined. The initial development is well justified and must be followed up.

In France, as in neighbouring countries, demand is tremendous. The French example of educational building programmes (schools, colleges, universities) is very edifying. Special papers will show you their consistency and the impressive results already achieved. The situation is no different in many other sectors — ministries, public services, hospitals, large housing estates, office blocks — all of them involving huge buildings which lend themselves very well to the use of mass-produced prefabricated components. One sector which deserves special attention is industrial building in general; rather neglected up to now, it is certainly one of the most promising fields of application for the new techniques.

I shall now deal in the second part of this paper with the question of supplies from the steel industry. I shall not touch upon heavy sections used for the framework and main structure, a field which has been thoroughly studied for many years: I shall deal purely with materials for the other parts of buildings, in particular with thin sheet, which is being predominantly used.

Here briefly, are some of its properties: production in large rolling mills, both hot and cold, erected according to the very latest progress in world technology; strict control of size and quality at all stages of production; outstanding suitability for complex forming, bending, stamping and welding operations. Strength for strength per surface unit it is the lightest of all materials. Together, these properties make thin sheet the ideal product for building. It has one drawback, however, which I must mention: it rusts when exposed to the atmosphere, but this familiar drawback has not hindered its tremendous expansion, and we are day by day becoming better enabled to deal with corrosion.

Thin sheet exposed to the atmosphere needs a more effective protection than just a coat of paint, because it is so thin. Stainless steel, which can be used without paint, claims special attention from this point of view, but its price limits its use in places where the question of money takes preference over necessity for high aesthetic quality or very high mechanical or chemical strength, but this is clearly not the case for most of the markets with which we are concerned here. The cost of enamelled sheet limits its use. Plastic-coated sheet is beginning to appear on the market, but apart from its high cost it is generally considered to need a previously galvanized base steel.

We thus arrive, almost of necessity, at galvanized sheet, which seems to be the most complete and economical answer to the different aspects of the problem. I should, however, make a number of points in this connection.

Black sheet for large steel structures is produced in the same mills and subjected to the same control procedures as sheet supplied to the motorcar manufacturing industry; that is to say that the base steel is, before galvanizing, one of the most highly developed of steel products, with the exception of alloy steels.

Galvanizing is similarly carried out in large continuous plants which with adequate control ensure perfect adherence and uniformity of thickness of the coating. Next come shaping and ribbing which have a twofold effect: the sheet is given resistance to bending stresses, and the product is moreover, in the circumstances, quite pleasing to the eye, quite different from the monotony of corrugated sheet panels.

Finally, from the very long strips, galvanized, ribbed portions are cut to their final sizes for use; these can be up to 12 metres long or longer according to the length of roof slope to be covered or the height of the walls to be clad. Handling on the site is thus considerably simplified and speeded up, and labour cost markedly reduced.

This is not all: from the purely aesthetic point of view, the satisfactory appearance of shaped and ribbed sheet enables it, in many cases, to be put in position without painting. The client can then leave it exposed for a year or two before painting. But it should be remembered that where used in conurbations and their immediate surroundings, aesthetic needs become greater and greater; it is here that one of the most characteristic developments of recent years has taken place, namely painting galvanized sheet in the works before delivery to the site.

Here are some interesting figures in this respect. Japan, the second largest producer of galvanized sheet in the world (almost 1,200,000 tons per annum), has been actively going ahead with this system for some years; the tonnage painted in the works has grown as follows:

1961	*1962*	*1963*	*1964* (estimated)
36,000 T	73,000 T	100,000 T	200,000 T

This growth is significant.

France, the third largest producer in the world, does not intend to lag behind: in 1965, the producers plan to supply builders with something like 20,000 tons of ribbed sheet painted at the works. This figure represents a substantial proportion of the total supplied to large builders, and it will increase further.

Finally, an important fact which must be emphasized, painting is not aimed solely at satisfying aesthetic requirements; it is a valuable aid to corrosion protection, especially in particularly corrosive atmospheres such as those met in urban and industrial centres.

What I have said about the French steel industry applies to all producers in our neighbouring countries. I do not doubt that details of the arrangements made within the Community as a whole will be received with great interest by the various bodies — both official and private — whose distinguished representations are with us today.

In particular, I would mention the eminent President of the International Architects' Union. His presence in the chair shows the interest which he and all his colleagues are taking in the problems raised by the rapid development of large building programmes; we shall always be pleased to have his opinions and suggestions.

The same is true of our respected friends from the Centre Scientifique et Technique du Bâtiment. Many of them will have found in my remarks an echo of certain desiderata they have been pressing upon the producers.

The heads of the designing and planning offices can also see that the new disciplines offer a vast field of application to their activities.

And last but not least I would say a word to the steel construction representatives. It is not my job to discuss the technical problems of application, prefabrication and industrialization. The results already achieved are remarkable. Improvements and amendments will doubtless always be needed, and this will be a daily task for the future. I shall confine my final remarks to recalling the close traditional links between the two industries of building and steelmaking.

I believe that the new concepts with their great prospects will only strengthen the co-operation between the two to their mutual advantage, aid the rational expansion of steel construction throughout the Community.

Jean-Baptiste ACHE

Technical Requirements of Industrialization

(Original text: French)

I listened with the keenest interest to Prof. Henn's important paper, and I noticed how he emphasized both the human aspect of these matters and their universality. That is why I should like to refer back to Prof. Petschnigg's paper at the plenary session yesterday.

I fear that this paper appeared to stress the opposition between form and function. This point of view seems dangerous to me, because it is liable to re-open what would become a primarily philosophical discussion on the conflict between the pleasing and the practical.

I also consider it dangerous to set engineers against architects: the rift that so embittered relations between them in the second half of the nineteenth century was an obstacle to progress.

Several references were made yesterday to the Eiffel Tower. May I recall here that Eiffel himself said, "My tower was modelled by the wind." The form we find so pleasing today thus arose entirely out of the tower's function, as a very tall pylon was not required to support anything, but had to withstand wind pressure.

The Eiffel Tower, though the landmark and symbol of the 1889 Exhibition and the assertion of man's mastery over matter and more particularly over metal, is not a model of industrialization, since like The Garabit viaduct it was made to specification, at the cost of 40,000 draughtsman-hours, with a precision allowing for each rivet-hole a tolerance of only 1/10 mm.

Industrialization however demands much else besides — the co-ordination of effort, the acceptance of various sacrifices. In a word, the industrialization of building is first and foremost a frame of mind.

When the first railway engines came into use, the stage-coach men robbed of a livelihood, tried to whip them out of town. In face of the inexorable advance of the industrialization of building, we must make sure that this attitude does not develop today.

However, it would be a distortion to think of industrialization purely in terms of prefabrication, although it is only the prefabrication of components that makes industrialization possible.

As Mr. Sittig has emphasized the advanced industrial manufacture of steel would appear to make it the ideal material for prefabrication.

If we look at the question from the consumer's standpoint, we find that there are really two consumers, the client and the architect. The client is concerned to get the premises he wants, industrial or residential, and to get them as economically as possible, that is, not only at minimum cost per unit of useful floor or cubic space, but with the minimum of delay, since the time taken in construction represents a tie-up of capital without return. The architect is concerned to give each structure an individual stamp, serving both to distinguish the building from its fellows and to characterise it as the designer's own.

Clearly, co-operation between engineers, metallurgists and architects must begin at a very early stage, in the actual designing of the components.

Equally clearly, the variety in building which Prof. Henn has rightly pointed out is so desirable can be achieved under an industrialized set-up only if the number of components available to consumers is sufficiently large; that is if the range of size standards is a wide one and, the range of quality standards covers all possible uses.

It follows, then, that

(1) A series of lengths must be established; corresponding to this there must be a combined series of varying sections;

- these sections should be available in a range of varying widths and thicknesses;
- so also should tubes, while as for sheet, the shapes and sizes of the different components made from it should be still more carefully tailored to the twin requirements of standardization and variety;

(2) It is quite out of the question for each individual manufacturer to conduct these studies on standards and size ranges on his own;

— what is needed is rather a series of national, or indeed European or international, standards, even if this means the specialization of different factories. The steel producers' freedom of operation would be thereby curtailed, but then, the engineers' already is, since they have long had to design their structures in accordance with the metals commercially available, and the architects will just have to accept an apparent limitation of theirs too.

And I mean "apparent", for they will soon see — this is the view of a number of architects — that in fact greater freedom may develop out of these relatively restrictive conditions.

On a site where industrialized methods are used, after the most careful planning on the basis of thorough studies by architects and engineers working in collaboration, buildings will go up which will be original and truly representative of our civilization because they have been designed from the outset in terms of the material employed.

But to my mind, unless all the conditions I have tried to indicate are fulfilled — organizational, material and intellectual — all we shall get will be structures possibly custom-built, certainly costly, pleasing more by good luck than good management, and prejudicial to industrialization, delaying its advent to the detriment both of efficiency and of aesthetic appeal.

Jean-Pierre HARDY

Young Architects and Industrialization

(Original text: French)

The problem of industrialization is an acute one for young architects and young Europeans like ourselves. We recognize its necessity and its benefits. We are trying to work out its implications for

— people in general;

— our profession;

— our training in technology.

First of all, we do not believe that there can really be such a thing as standard programmes with regard to industrialization. There can be permanent elements in the programmes, such as number of pupils per class-room, sanitation in housing, etc., but has everything been accomplished when the programme has been adhered to?

Can it be said that the architect is not required?

We claim our right, not to compete but to co-operate with the technician.

The architect is neither an artisan nor a dreamer: he is a technician in the science of humanity, an inventor.

We were told this morning that the architect was acceptable for buildings in the national education programme since he was the "prisoner" of accepted standards. But did he ever help to create these standards? We accept a discipline if we think it is the right one.

M. Petschnigg made too sharp a distinction yesterday between the aesthetic and the technidcal. We believe that the two things must go hand in hand, not turn their back on each other. Moreover, technical matters are for us not merely a question of buildings or panelling, operations or raw material or cost prices, but also of human beings (mental health, social welfare) and environment (town planning, geography, economics).

It is not merely a question of finding new outlets for raw materials, but also of putting them to the service of man — not the reverse.

As regards housing, a very topical question, there is talk of manufacturing dwelling units on mass production lines, like cars. Fair enough. But while the car manufacturers can afford to produce hundreds of thousands of cars without worrying about questions of parking, motorways and bridges, it is our duty to state that it would be criminal to produce dwelling units without thinking of their integration within the framework of a town. Be warned! A town consists of dwellings, horizontal or vertical, and of streets, cultural, commercial and recreational centres, social centres, cinemas and public houses. It is not enough to provide shelter for individuals; we must organize communities in which everyone can find his joie de vivre.

Nobody here has so far uttered the words "town planning". Yet it is the townplanner's job to decide the lines on which the units are to be organized in relation to each other, and to plan the open spaces and the developable areas. The town planner must not be consulted at the end, when it is too late: it is at the very beginning that he, and later the architect, must be called in to work out the programme, and the technical men to deal with the technical problems, and then all of them together to complete the undertaking. This is the only way to success.

In conclusion I should like to talk of the problems of education, of the popularization of technology, problems which very directly affect us. Building with steel presupposes very thorough advance planning and a high degree of precision in the planning stage. Our schools of architecture give us a general education about materials, but very little about the methods of working with them and nothing at all about actual practice.

We need to touch the materials, to see the sites and to meet the technicians. We should ask the technicians to come to us, to insist on contact with the schools in spite of the slackness of the authorities and the teaching staff.

Architects and engineers can no longer ignore or oppose one another. European-level contacts between architects, engineers and the industries must be arranged far more frequently. Mutual understanding must be sought in order to pave the way for true teamwork.

Romke DE VRIES

Industrialization and Human Problems

(Original text: Dutch)

I should like to begin by expressing my appreciation as an architect to Professor Henn for the many points he has so ably introduced from the topic for discussion in this Working Party.

It is unnecessary to harbour any hostility to steel in order to feel oneself somewhat menaced in this company by a variation on an old theme: for steel it is prefabrication or death — an attitude that is perhaps not unconnected with the decline in steel consumption that has been noted by the E.C.S.C. and that has taken place in spite of the increase in the number of building projects and of the fact that until only a short time ago it was difficult to obtain prompt delivery even of a single steel girder.

The official addresses and the general discussions at this Congress both reveal a certain pattern of approach according to which the metalworking industry assumes the right to regard itself as superior to the more traditional building industry. This assumption is based primarily on the three-thousands of one inch accuracy and indoor working conditions so characteristic of the steel construction industry, as contrasted with the relative inaccuracy and unsheltered working conditions of the building trade using traditional stone, wood and concrete as its raw materials. This biased attitude of superiority is accompanied by an exageration of the technical demands of the manufacturing processes to which the use of steel gives rise, and an under-estimation of the countless other factors which must of necessity determine the ultimate form of the structure, particularly in the case of housing and of school construction.

A French manufacturer of prefabricated houses has frankly admitted that although he failed originally to take account of this great complexity of factors, apart from the purely technical aspects of the manufacturing process, he too was ultimately led to them empirically just like the traditional builders, architects and town-planners. I feel therefore (and I shall return to this point later in greater detail), that entrepreneurs in the field of prefabrication, Dutch as well as others, in their introductory speeches before this Congress, tended to minimize the importance of certain definite and specific aspects of housing and school construction. They considered the execution of building projects by the precision-manufacturing industry with the addition of architectural know-how (quite apart from know-how in the art of building) to be necessary to their superior execution by the building trade, even with the meanwhile greatly increased industrial know-how (Hageman).

Both houses and schools, after they have been completed eventually form part of so many wholly or partially overlapping social inter-relationships that the manufacturing process, important as it is, constitutes only a very small part of the whole pattern in terms of time, space and organization.

In connection with the foregoing, I feel that I must protest, against the biased and systematic tendency to describe current traditional building projects as quite the most grim and badly organized projects in the world of today, carried out in the rain and cold by badly equipped, poorly-clad workmen (Mr. Sittig), in contrast to the prefabrication process always represented as carried out under ideal, sensible and scientifically correct conditions by enthusiastic young executive types in white jackets in brilliantly lighted and overheated factories.

This is not a fair comparison. One might with as much (or as little) justification compare a well-managed building project with a badly-organized factory — after all, how many well-managed building projects are not constantly held up because of the failure by one or more of the supplying industries to honour agreed delivery dates ?

The organization of a modern building project is often excellent. Even though it may appear that this organization has to be set up afresh every time a new project is started, it is basically ever present, like that of a great circus (such as Barnum and Bailey's). To the casual observer, this organization exists only from the moment when the first material results become visible on the building site. It is perhaps for this reason that an industry based on and accustomed to a single and constantly available indoor working site, *i.e.* the factory, will never fully understand either the nature of the organization of an open-air building project or, much less, the preconditions and requirements that the preparation for that building project, as a real property asset, has had to satisfy in society before it could proceed to the stage of execution (large investors paid no money for a good concrete frame when there wos any suggestion of "prefabrication").

The well-managed building project may be validly contrasted with the inefficient or inadequately efficient factory; this does not necessarily mean an inefficiently managed factory: its efficiency may well be the maximum possible at a given time and in o given place, this maximum being determined by historical factors, in the same way and to the same degree as that of the building industry, allegedly hondicapped by its artisan origins.

That a certain reservation is justifiable on our part with respect to the changing over by shipbuilders to the prefabrication of buildings is illustrated by the following very clear example of how such a change-over may be affected by historical factors. In 1947, the Bristol Aircraft Company was compelled to cease production of prefabricated aluminium buildings that it had only recently started. Since the Company possessed for this purpose the highly perfected and extensive ossembly line equipment of its wartime Blenheim aircraft plant, this disaster was clearly attributable not to the equipment but to the human shortcomings in its operation. This does not mean that these human shortcomings (laziness, go-slow working, smoking on the job against orders, etc.) did not exist in wartime aircraft production plants; on the controry, it is probable that they first began to be tolerated at that time, under the force of circumstances, the profit margin on the aircraft being high enough to permit the consequent losses to be absorbed.

This was no longer the case, however, when the Company entered the naturally competitive field of postwar housing production for the masses. It found itself unable to compete, despite a certain measure of Government protection and despite the advantages of possessing a highly developed plant, which had appeared to present it with a unique opportunity.

Consequently, the question remains as to whether the technical possibilities will in fact always be adequately supported by the financial possibilities when a manufacturer embarks on the production of houses or of components for houses in all the wide variety that is essential if human dwellings are to be built that are complete in every respect. "Complete" implies, on the one hand, culturally complete, *i.e.* more than satisfactory both from the point of view of town-planning and for living, working and recreation, soundly designed from the points of view of physical and mental well-being, and adapted to the needs of the rising generation from the point of view of opportunities for education and experience; and on the other hand, aesthetically complete, *i.e.* that the buildings are actively and consciously satisfying in themselves, in terms of form and space, from the points of view not only of composition and urban spatial planning but also of the general desire for beauty. There is a danger that the only enterprises to succeed will be those turning out entire dwellings in only one or two designs and on a large scale. This would lead to a standardization of those few types (such as in fact is already taking place), which would seem to be fatal to any further healthy development of housing and estate design.

This danger must be clearly pointed out in the conclusions of this Congress.

A further danger is that prefabrication and high-speed assembly, as aims in themselves in total disregard of all other factors, may become as it were a toy of the engineers, irrespective of the type of school or house concerned and regardless of the long-term cultural objectives.

From one's observotions during the Congress one cannot help wondering whether, quite apart from the questions of the direct manufocturing process, adequate attention is being paid in the production teams to the culturol ospects of our time: to the effects of increasing scientific knowledge and technological advances on automotion and the increased leisure time that this implies, to its consequences for the development of the individuol and its effects on housing design needs in the none too distant future, when the majority of the average families of today will have made their way up to culturally well-developed environments.

In one sense, the technical perfection desired by the engineers is intellectually feared by others, including Mumford and the late Dr. Rietveld, an architect who delivered lectures on the subject of "How do we protect our dwellings against middle-class conventionolization and technical perfection" — in which the naming of these two concepts in the same breath leaves no room for doubt as to his feelings.

On this point, a factor that immediately arises in considering the question of relative cost price and technical perfection is that, according to their different cultural attitudes and orientations, one person will attach greater importance to more space while another will be prepared even to sacrifice space for greater technical perfection, when the cost of both these advantages together is prohibitive. During the recent experimental dwelling prize competition of the *Bond van Nederlandse Architecten* (Association of Dutch Architects), for which, as a member of the panel of judges, I helped to draw up the programme, particular emphasis was laid on these two possibilities for the spending of the additional 5% promised by the Minister for the successful experiment.

Other aspects that it could be dangerous to neglect and that might therefore also have received the attention of this Working Party include the necessity, to which attention has been drawn by C.I.A.M., I.U.A. and U.N.E.S.C.O., for efforts be made to solve the problem of housing construction by limiting the real property element to basic structures of relatively long useful life that can be readily adapted with changing times to the prevailing philosophy of living, both as to form and as to size, by means of inter-changeable ancillary parts of shorter useful life. Although dedicated steel constructors advocate in opposition to this the demolition and replacement, within a relatively short period, of the *entire* building, this view appears to have had little impact so far.

Another factor that emerges from international exchanges of views between architects is that of the regional character.

derived from the location of the building site, that must distinguish every truly functional architecture and that immediately becomes evident when functionally similar buildings have to be designed, from the same architectural starting points, for erection on sites in completely dissimilar countries (Gropius). For this reason alone, quite apart from others, it will be clear that the prefabrication of dwellings in a similar manner and on a similar scale to the mass production of automobiles is out of the question.

In my view it must be concluded from the foregoing that it is essential not only that architects, town-planners and sociologists participate in study groups and production teams but also that they be incorporated without delay into the study of this subject at international level.

However desirable speed may be in building the dwellings and schools needed by society, and however useful steel can be to us in that connection, we must first decide which things come first, *e.g.* whether we prefer an airy but rather hollow-sounding steel building which, though it is to last for fifty years, is completed five months sooner, or a somewhat weightier traditional building which, although not perhaps the last word technically in other respects, is rather less noisy and therefore provides a rather more restful environment in which to live and work.

Notwithstanding the nature of the main topic of discussion at the meeting, we shall have to decide what are the requirements that the infrastructure for the mental and physical development of the individual in society must satisfy and which materials are best suited for that purpose, not, of course, omitting to take into account in each particular case the possibility of construction wholly or partially of steel. The converse approach, to assume as a starting point that everything will be built of steel and then to consider whether the consequences will be harmful in a particular case, would appear to me to be unfruitful.

Bernard C. L. CHRISTIAENS

Certain Social Problems of Industrialization

(Original text: Dutch)

I should like to say a few words about full scale industrialization based on prior standardization and culminating in automation, as seen by a Belgian private architect.

There are a number of basic negative aspects which can be regarded as problems ; to my mind, if these can be solved, we shall be taking the first major step necessary to fully industrialized building.

The first problem is the land policy. How do we build in our cities at present? By individual replacement of buildings between existing old party walls, instead of rebuilding whole blocks and whole quarters based on proper principles of town planning and architecture. We shall never achieve industrialization that way. And how do we build in the country ? Often on poorly chosen, scattered sites, difficult to obtain by compulsory purchase and involving all manner of personal and political influences.

The conclusion is that we need a different land policy. It has got to come some time, but it will take years and years. I would say that full industrialization of building begins with the land policy and not with standardization.

The second problem is that standardization, and consequently industrialization and automation, can involve serious dangers if the projects are not in the right hands. For all this will need to be top quality — the ultimate objective, the architectural design. That will require a great deal of detailed study and a great deal of study means a great deal of expense. Our designers are not paid enough to devote the necessary amount of study to the designs: consequently, no progress in building. How is this state of affairs to be remedied ? (What I am saying may apply to other countries too.) I think what is needed is protection of the status of the architectural profession, the establishment of an Architects' professional association. Actually, Belgium did recently institute an *Ordre des Architectes*, but we are still waiting for it to come into effect.

A third problem is that in a matter of such proportions as the full industrialization of building, the work of the individual designer has little or no influence. We need a team spirit, and that is something which with few exceptions, has not so far been found in our profession. I think the architectural colleges could do an enormous amount to inculcate this sense of co-operation at an early stage in the designers of the future, and co-operation with all branches of building activity.

Conclusion

My point is that industrialization of building is a very radical and far-reaching thing. We in Belgium shall need to go carefully into many preliminary aspects before we can see clearly how this can best be done and with what kind of materials.

I know that some other countries are well advanced in the industrialization of building, and indeed in the use of steel — among others Britain, with the ,,CLASP'' system, in which steel is the main element.

In conclusion, I would say that we in Belgium are nowhere near industrialized building, and shall have to tackle a number of preliminary problems first.

Mario ROGGERO

Joint Statement of the Italian Architects

(Original text: Italian)

I am speaking on behalf of all the Italian architects who are present at this meeting seeking not so much to define our position regarding the use of steel in building, (this is a highly complex matter which cannot be dealt with in a few sentences or fully discussed at one meeting,) but rather to emphasize strongly that organized, regular meetings should be established between the different branches represented at this Congress.

The Congress has enabled us to state our views and attitudes on this subject, and the architects present have responded as conscientiously as they could, even though they were not all in complete agreement, they have endeavoured to offer the Congress a number of practical solutions in order to stimulate an exchange of ideas with all the groups who deal with the use of steel in building.

Therefore on behalf of my colleagues, I would like to read the following joint statement:

"Whereas the European Coal and Steel Community has convened economists, producers, technical specialists and designers with the object of promoting the increased use of steel in building, we, the undersigned, the architects and engineers of the Italian delegation to the Congress, would like to call the attention of the Bureau to the following points:

)1.) There is a substantial likelihood of concordance between the interests of the groups represented in the Coal and Steel Community and the Italian designers, on the more extensive use of steel in line with the social, economic and technological trend in their countries.

2.) As it is proper that each Community country should indicate its own priority development plans, it is noted that a major operational programme is now in hand in Italy concerning large urban structures, and new Government schemes are impending for residential, school and hospital buildings, in which there would be considerable scope for the use of steel.

This being so, we trust

— that the High Authority will give every encouragement to efforts being made to enhance the advantages of using steel (notably by aid to University research, technological education and occupational training);

— that an appropriate programme of experimental, industrial-scale and applied research will be instituted;

— that future meetings will be more specialized, for instance taking the form, at a national level, of study groups, and at an international level of specialist sectoral committees. In the view of the undersigned, the object of such meetings should be to compare notes on problems of full industrialization, both "open" and "closed", of partial industrialization relating to particular components, of composite building techniques and of the popularization of steel utilization techniques; these occasions to be organized and promoted as part of the institutional work of the High Authority.

Subscribing to this Italian statement are the following architects and engineers: Albini, Bagatti-Valsecchi, Chiaia, Ciconcelli, Ceragioli, Daneri, Fagnoni, Fiorentino, Gardella, Necchi-Rusconi, Gugliermella, Helg, Manfredini, Matteoli, Moretti, Pazarasa, Perugini, Piccinato, Rutelli, Valori, Viganò, Vitale, and Roggero."

Stéphane DU CHÂTEAU

Steel — The Material for Architecture

(Original text: French)

The great strength and energy of steel, the remarkable mechanical properties of circular hollow sections, the development of welding techniques, the efficient methods of protection against corrosian provided by metallization and galvanizing, the economy of weight and the aesthetic qualities, all fully justify the marked preference for this great material, permitting advanced designs whose limits can be specified exactly, whilst offering a guarantee of durability, the first essential for all architectural work.

Steel engenders designs which are peculiar to themselves, which ensure flexibility and create their own style by their simplicity of lines, which reproduce the systems of natural structures and whose stability can be scientifically proved.

These new possibilities in design promote similar trends in the conception of building : beyond the traditional, and it is possible to imagine quite freely "three-dimensional structures" in space.

So we have entered a new era which tends to plan and close its spaces by direct methods of construction alone. The structure is integrated into the architecture, the character of which it determines as much by the principles of stability as by the geometrical nature of its frame or its structural depth, its aesthetic values are recognized and sought for as a material for flexible composition.

These new materials also bring to architecture the creative means whose aesthetic values are enriched by the intellectual contribution accorded to the art at its highest level.

As the beauty of marble flatters and attracts the eye, a beautiful structure astonishes, stimulates, provokes thought, makes itself understood, causes the viewer to think, gives rise to further ideas and awakes the intelligence.

It is fertile and educational.

In the churches, schools, and factories, it brings this particular quality which honours the person it shelters: when it shelters with the grace and dignity which are the marks of true building and which becomes architecture.

Thus, the consideration of economy ceases to be the determining factor. To turn to account the qualities of a superior standard of work, adequate means become necessary.

To find and impose these values, the architect must be in possession of all means of designing and fulfilling the architectural spaces, with full freedom of choice, of technical means and decisive materials for this aspect of the work.

It follows, therefore, that he must know all these techniques, but it is scarcely conceivable that he knows them thoroughly. To apply them he will call upon specialists, engineers and contractors who, for their part, will have to be introduced to the problems of the architectural creation and brought into collaboration, in order that a dialogue of constructive understanding may take place from the moment of conception of the dimensions of the work to be built, to bring to it "The skilful and magnificent play of light and shadow" of Le Corbusier or, like Auguste Perret "Chanter les points d'appui".

Jacques BENDER

All-Steel Industrialized Building and Architecture

(Original text: French)

Years of study and experience have convinced me of this : industrialization means steel. Frames, floors, roofs and ceilings, external and internal walls, furnishings and fitments — the building of the future will be steel throughout, perfectly machined, finished and protected, long-lasting. With its technological advantages and its adaptability to different purposes, steel lends itself to building just as it does, and indeed a good deal more abundantly than it does, to the construction of ships, railway carriages, or motor cars. All building components will be mass-produced with precision machines and tools which were not available to the builders of the past. This will make for better and better quality; such is the rule of industry. (1)

Another reason for "all-steel building" is the very much shorter time taken in construction and also the lower costs.

Erection does not necessitate the pouring of masses of concrete, but only of assembling and bolting — operations which can continue in all weathers. The occupant does not have to hammer nails : he just moves a bolt or a magnet, he washes his house as he washes his car.

Furthermore, manufacturers will mass-produce buildings : They will stock sets making up whole structures, all consisting of standard, interchangeable components. In the future most building will be fully industrialized. One cannot put back the clock. What is needed is organisation designed to achieve mass-production of the houses for which the people are waiting.

This necessary revolution is obviously going to disrupt completely a number of outdated present arrangements. A five-year or even a ten-year plan will be required after the first experiments before full mass production is achieved.

What is industrialization ?

It is essential at this stage to define precisely what constitutes industrialized building : this is necessary for the drawing-up and the allocation of the programmes, for these can only be entrusted to firms of contractors operating on a properly organized industrial basis.

Industrialization of building may perhaps be described as having four aspects : it is a process, a fabricating method, an assembly technique and an integrated enterprise organization.

(1) The building process must comprise the following structural components : frame, floors, roofs, external and internal walls, ceilings, staircases, and must include all the trades involved. (2, 3, 4)

(2) All these components must be entirely machine-made in specialized factories with accurate mechanical equipment, turning out large quantities of perfectly finished products on the production line. The production ranges represent a known set of specifications and are subjected to repetitive check-ups at all production stages; the supervisory offices will conduct checks within the factories. (5) (See page 453)

(3) On site, all the prefabricated components must be assembled by streamlined modern methods, so that all types of buildings may be planned and constructed in accordance with the rules of architecture and without the need for auxiliary trades.

(4) The industrial contractors must have a permanent staff of specialist engineers qualified in all techniques. These will be responsible for planning and supervising all sites, with their civil-engineering projects and outside operations.

Research

In industrialized architecture, as in industry, research is indispensable to progress.

By this I mean architectural research in all forms — progress on industrial techniques, new applications, integration of modern components on different sites, the creation of functional architecture in line with the new requirements of our time, insistence on quality in internal fittings, finishing, line, colour and so on. (6)

I would recommend the establishment of an Industrialized Architecture Research Institute, where architects could work with the materials and services of one or more factories, and be enabled to compare notes with professional colleagues on the type of structure they are designing and on the practical details involved.

Side by side with this research could be consultation with a variety of specialists from the sphere of art—painters, sculptors, landscape gardeners—thanks to whom our frontages, the colours we build in, the parks and gardens for man's enjoyment, could be made to express a new delight in living.

The European iron and steel industry, and all the firms manufacturing with steel, would combine in this drive and give a new look to building.

The team-work involved

How will the industrialist and the architect collaborate ?

Despite initial difficulties, they are bound to come steadily to one another. At any rate, I believe they are, because it is necessary and inevitable that they should. The architect cannot live outside his period, and he cannot live on his own. His contacts with the industrialist cannot be sporadic and occasional. The two must get on one another's wavelength now, and remain there.

And what resources industry can make available to the architect if he will only make the effort to come into line with it! What we have to do—and do at once—is to establish team-work between them, by appealing to the rising generation of architects and of industry to be inventive, imaginative, and forward-looking.

Industrialization demands that all the problems involved in achieving functional architecture and modern building methods be clearly stated and studied. Short-term programmes are not enough. It is up to the all-round planning bureaux to deal with the problems. We can be sure they will be dealt with. The task of the industrialist, the engineer and the architect is to build in partnership for the future of mankind.

Our various experiments have yielded the following preliminary results. A technical college for 800 students, comprising 17,500 sq.m. (62,790 sq.ft.) of floor space and nine buildings of three to five storeys was erected in five months, using 400 tons of structural steel and 1,000 tons of galvanized sheet.

We designed the following types :

Type of building	Useful floor space	Fabricating time	Building time	Sheet used (metric tons)	Structural steel used (metric tons)
Workshop	3000 sq.m.	4 days	15 days	65	51
College bldg. (1 storey)	1500 sq.m.	8 days	21 days	72	33
College bldg. (2 storeys)	2600 sq.m.	15 days	30 days	130	65

The times given are for all-steel buildings (7) flow-line-produced at the factory and assembled on the site in accordance with a carefully-worked-out programme.

The costs were 10% lower than they would have been had traditional methods been used, and savings on foundation work and cost alterations amounted to 10-12%, giving a total saving of 20-22%

Now, if French housing requirements average 125 million sq.ft. of space a year, we may reckon housing requirements in the European Community at something like 360 million. Surely it is not going too far to suggest that in a few years' time a quarter of this building — 90 million sq.ft. — will be constructed and fitted in steel.

That would give an annual consumption of 1,500,000 metric tons of sheet and 650,000 of structural steel—representing an annual turnover of $ 300 to 400,000,000 for the iron and steel industry alone, and a corresponding figure for the manufacturing industries, This possibility is within the European steel industry's grasp. It is striking to note that in France, at our prompting, steel has secured 10% of school building, *without* a co-ordinated national programme.

In the course of our activities, we have embarked on preparatory studies with architects, and have obtained a first series of most valuable results with regard to the composition of bills of quantities, colours, finishing, and the devising of new components such as eaves, balconies, recesses and so on. (8)

In addition to studying the initial designs with the architects, we have established the first *after-sales service* in connection with building. In due course the after-sales service bureaux will become area concessionnaires, and ultimately builder-assembler groups equipped with the very latest facilities, amounting to what might be termed "building service stations".

Upon this type of organization all-steel industrialized building is based.

These experimental ventures seem to make the beginning of a complete change in building techniques. They serve to clarify the real problems of industrialized construction, which the steel industry will need to solve for the benefit of the population before the end of the century.

It would therefore be worth including courses on industrialized steel construction in the syllabuses of the architectural, civil-engineering and technical colleges. Students should be enabled to learn by visiting factories engaged in the production of building components, and also where appropriate travelling exhibitions sponsored by the High Authority.

Description of photographs

1 — All steel façade of five-storey building.

2 — Assembling of stair- and floor components of a five-storey building.
Façade and partition sections added during erection may complete the structure.

3 — Steel floor sections ready for assemblage.

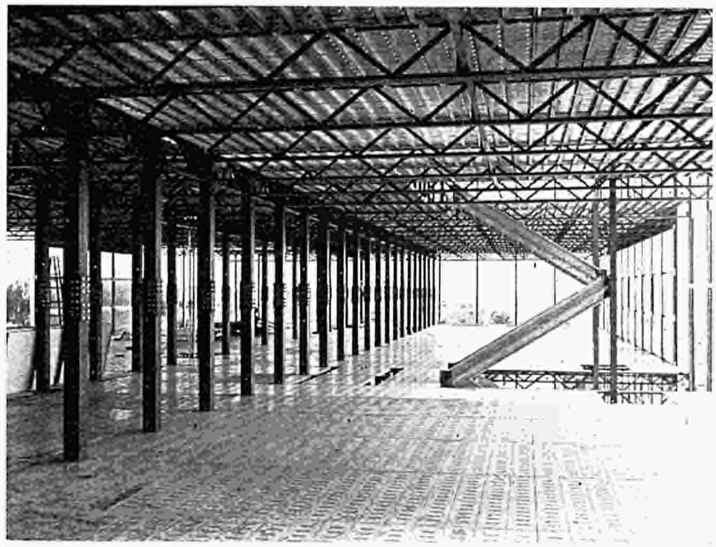
4 — Preformed sheet metal floor sections assembled simultaneously with other components.

5 — Finishing of flow-line produced window frames (preformed metal sheet) after painting.

6 — College building at Faurmies (Nord). Façade with eaves and wall recesses.

7 — Exterior view of college at Limeil-Brévannes (Seine-et-Oise), constructed in two month (1964).

8 — College buildings at Montargis (France), dormitories, school and administration buildings.



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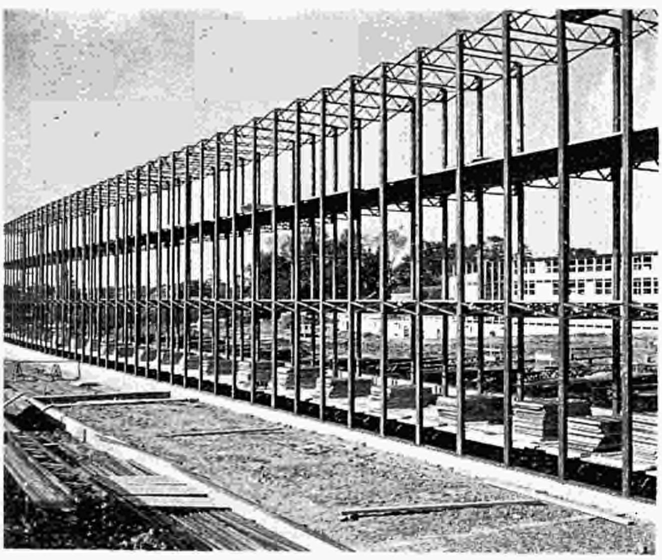

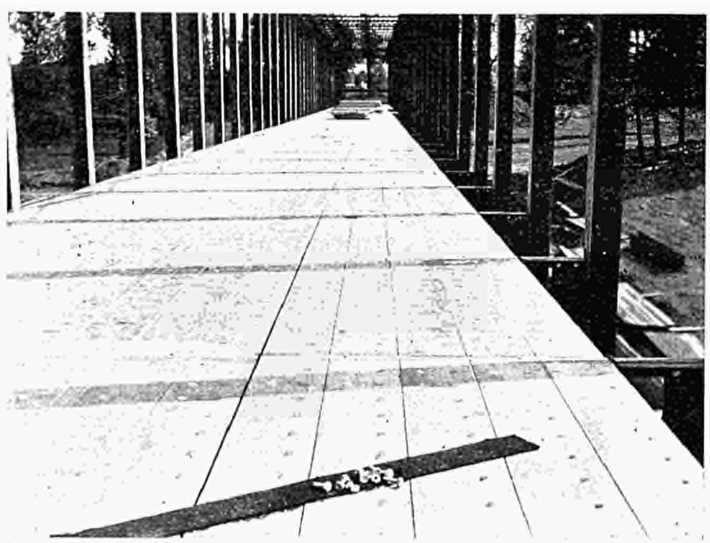

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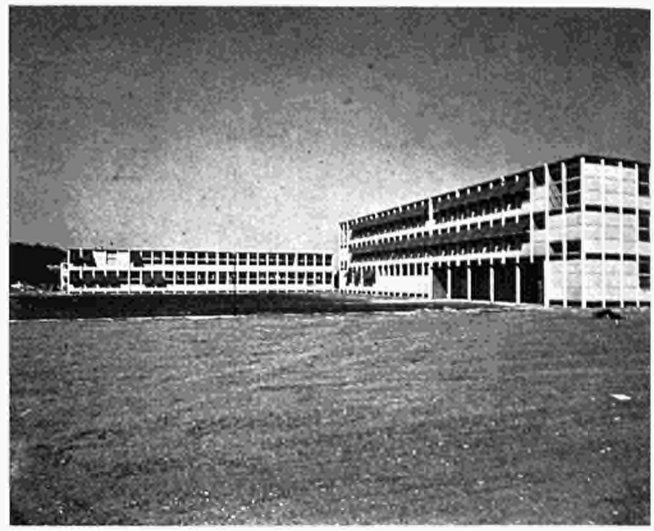

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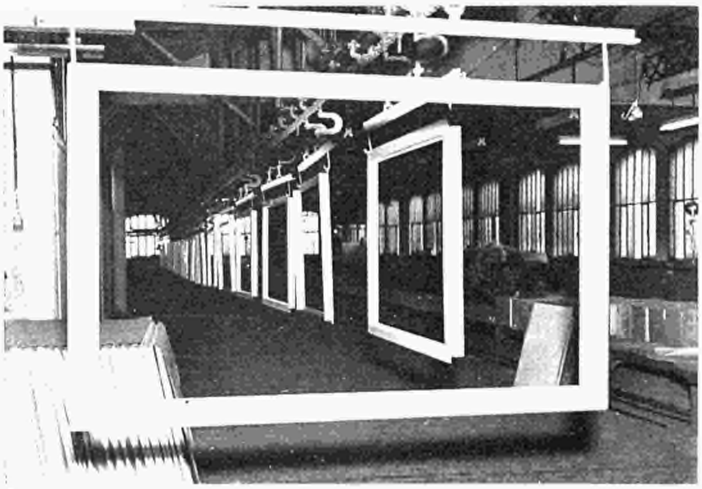

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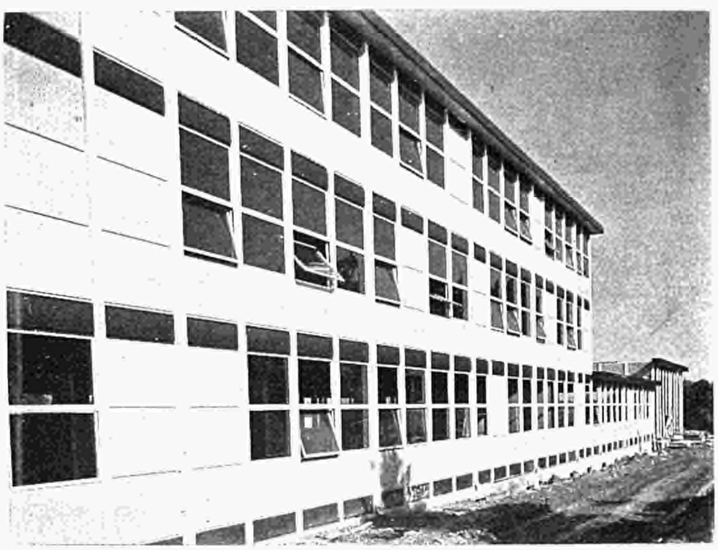

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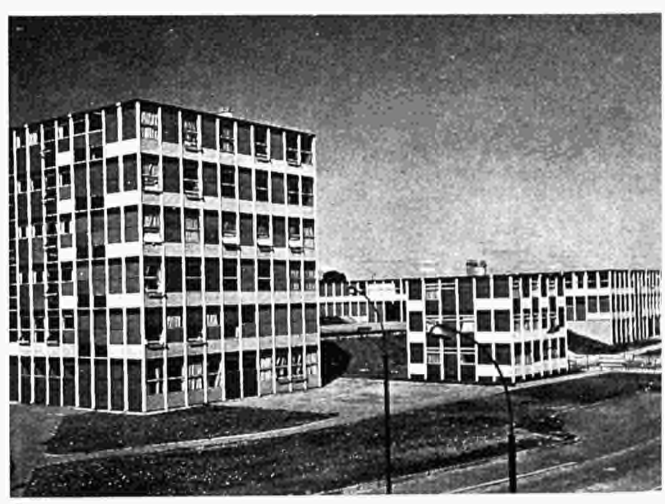

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Hans-Jürgen DANKERT

"Unit Building System"

(Original text: German)

I should like to take up the point which Professor Henn has just made concerning the ,,open system" for the planning of a building. It has already been mentioned that the lack of ,,systematization of dimensions" largely precludes prefabrication at the present time, certainly where large buildings are concerned. You may therefore be interested to hear how Krupp's, in Germany, have for the past two years or so, been very successfully erecting buildings of various kinds using a fully individual type of prefabricated unit system. This system is based on the maximum possible use of the assembly method of building, enabling completion times to be shortened and making the work completey independent of weather conditions.

In the planning of a building from the engineering and structural aspect one naturally endeavours to ensure that as many parts as possible are identical. In this way planning and production costs are reduced. The use of identical structural elements in different buildings usually proves impracticable, for reasons which have already been mentioned at this meeting. Yet when the engineering aspects of a number of such buildings are planned, in co-operation with various independent architects, and when we, as main contractors, erect them in readiness for occupation, we find that there is a certain correlation between the various structures. It is true that they do not, as I have said before, consist of the same parts, but they are based on the same principles and on the same well-tried solutions. The structural problems — such as those involved in designing a wall that will be both acoustically and aesthetically satisfactory, or in connecting it to a ceiling — continually recur in the same basic form, subject to certain variations. The factors that do differ from one case to another are the demands made on the individual parts of the structure; I am thinking here of flooring, ceilings and the means for protection from the elements.

Now it is perfectly possible for the ceiling of a school or administrative building to be comparable in principle to that of a hospital. The important point here is not whether the building is a hospital or a school, but whether or not the premises are to be air-conditioned, or whether or not a false ceiling is required.

A factor common to all these buildings is that the load-carrying framework is of steel and that this valuable material is, moreover, only used, where we find its use to be economical.

The building material industry produces a sufficient variety of materials suitable for use in the prefabricated unit system which I have mentioned. I am thinking here of prefabricated concrete slabs, plasterboard panels, false ceilings, etc.

It has been our experience that the use of steel for the supporting structure of a building is economically more satisfactory, not only from the point of view of the programming but also it ensures greater accuracy of fit and thus simplifies the completion of the main load-carrying part of the structure, even though a cost analysis of the bare framework appears initially to indicate that in situ concrete would offer certain advantages.

I have already mentioned that in the use of our prefabricated unit system we adapt ourselves to individual wishes and requirements and therefore re-apply, as frequently as possible, certain well-tried solutions in matters of detail — in varying combinations. In order to ensure that, in steel building, our activity is equally adapted, as regards the particular load distribution desired, to each set of conditions arising, and that the possible spans are utilized to the full, we endeavour to standardize the connections of our girders, but not their lengths.

It has been my experience that the prefabricated unit system of which I have just given a brief description, and which is already being widely used by Krupp, proves extremely suitable for buildings of appropriate size categories. If the planning architect, who of course has to take account of this method of building in the sphere of activity for which he is responsible, is a good architect, he will succeed in devising a satisfactory solution based on the system under discussion, as opposed to conventional systems.

As has already been pointed out by Professor Henn, the use of a prefabricated unit system of this kind will naturally be simplified where a main contractor erects a building in readiness for occupation. The main contractor concerned knows all the building materials available on the market, and has available all the data required for optimum co-ordination at the planning stage and, in particular, at the erection stage. As he himself plays a major part in the erection of the bare frame, and is also responsible, together with sub-contractors, for the completion of the whole of the rest of the building, he exerts from the very start, a considerable influence, on all that takes place on the building site. It is true that a main contractor is not necessarily indispensable for a prefabricated unit system, but it is by this means that the advantages obtainable can best be utilized to the full.

Professor Henn mentioned that when a building is planned by a constructional steelwork firm it will always be of steel and that one planned by a concrete firm will invariably turn out to be of concrete. Thank Heaven many of our competitors make the mistake of wanting to produce everything of "their" material wherever possible. I feel that we are closer to the ideal when the economically and technically satisfactory building materials are employed in combination with one another. Competition between steel and concrete will be enough to ensure that no unwelcome standardization in the choice of building materials will come about.

As a steel man, I am happy that we have received contracts for a whole series of buildings which are to be erected by the process which I have described but which, if their planning and execution had followed conventional lines, would quite certainly have gone up in concrete.

In addition to the technical and economic aspects on which we have touched, a main contractor naturally has a good commercial chance of being able to quote a fixed price and a guaranteed completion date when tendering for a building in readiness for occupation. He also guarantees that the building will perform its function and accepts responsibility for the structure as a whole. In my opinion the risk for the main contractor is considerably reduced if he bases his tender on a prefabricated unit system.

As already stated however, the situation in Germany is such that this prefabricated unit system has to be adaptable.

Eugène MARZIN

Industrialized School Construction

(Original text: French)

The increase in population since the war and the raising of the school-leaving age necessitate the construction in France each year of a large number of additional school buildings.

In 1964, more than eleven million pupils went to school, some 240,000 more than in 1963 for all the various types of school.

It is in the field of secondary and higher education that the building drive has been most marked.

Many steel fabricators have played an active part in this drive and some of them have joined forces, at the invitation of the education authorities, to build 45 grammar schools and colleges, as well as a faculty.

They have also provided a proof that structural steelwork can be used to produce high-quality schools economically and in a short space of time.

Three examples which are particularly significant are given here.

Technical College at Goussainville (S. & O.) for 432 pupils

This college (1) had to be constructed in 9 months, but, in accordance with the order passed in June 1964, the first part comprising the classrooms and the kitchen and dining-hall (shown in photograph 1) had to be ready in five months.

Construction is being carried out by means of light steel elements, standard and interchangeable with a module of 1.80 m. (71 in.) which, when assembled, provide a prefabricated building of real character.

Main characteristics

The steel frames are made from sections delivered directly from the mills, and only require cutting and punching, which can easily be done by mass-production methods in the shops.

Steel in the form of galvanized sheet is also used for the floors and roofing, while vitreous-enamelled sheet is used for the external face of the curtain walling, as shown in photograph 2.

The variety of enamelled panels in multiple dimensions of 30 cm. (12 in.) and the wide choice of colours give the architect every opportunity for aesthetic expression. Maintenance costs are negligible. In addition, acoustics and air conditioning are perfect.

The buildings were started in June and will be finished in October, ahead of schedule.

Secondary School at Limeil-Brevannes (S. & O.) for 1200 pupils

This consists of an extension to an existing building.

The technical considerations are similar to those in the previous example, but the curtain walls and the partitions are entirely in galvanized and painted steel sheet, as shown in photograph 3.

Although the classrooms are more extensive here than in the previous example, being on three floors, they will also be finished within the contract period. (4)

In both these examples the speed of erection is the result of long experience of the constructional system and of perfect site organization.

Faculty of Arts and Humanities, Paris

This six-storey building, which is built in conventional structural steelwork, extends over a length exceeding 100 m., as shown in photograph 5.

Steel is also used for the floors, ceilings, cladding, fittings and stairs.

The repetition of a very large number of similar elements, base on a module of 1.75 m. (69 in.) justified the application of industrialized methods, not only in the shop but also on the site (6).

A carefully drawn-up work programme has been scrupulously followed by the diverse trades involved. The steel fabricator is playing the leading role and the project will be completed in less than eight months so that the Faculty will be opened as planned on 1st November (7).

Time-table on the site :

1st February	Demolition of old buildings.
March-April	Excavations and foundations.
22nd May	Erection of first frame.
21st August	Four-fifths of the curtain walls fixed in position, several rooms already completed.
1st November	The building will be entirely finished.

Description of photographs

1 — Technical college at Goussainville (S. & O.) : July 29, 1964.

2 — Technical college at Goussainville (S. & O.) : September 22, 1964.

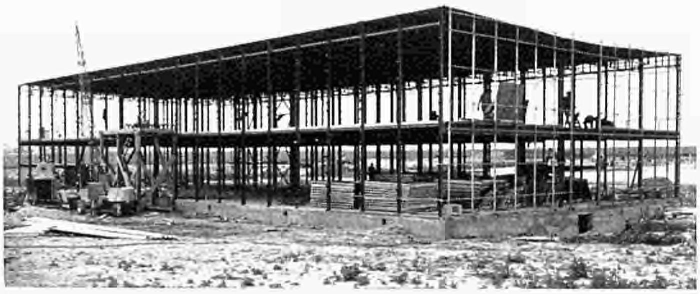
3 — Secondary school at Limeil-Brevannes (S. & O.) : September 22, 1964.

4 — Secondary school at Limeil-Brevannes (S. & O.) : September 22, 1964.

5 — Faculty of arts and humanities, Paris : Erection of the steel construction started : May 22, 1964, Photograph was taken June 12, 1964.

6 — Faculty of arts and humanities, Paris : August 5, 1964.

7 — Faculty of arts and humanities, Paris : August 18, 1964. In the foreground : steel construction of the auditorium with 400 seats.



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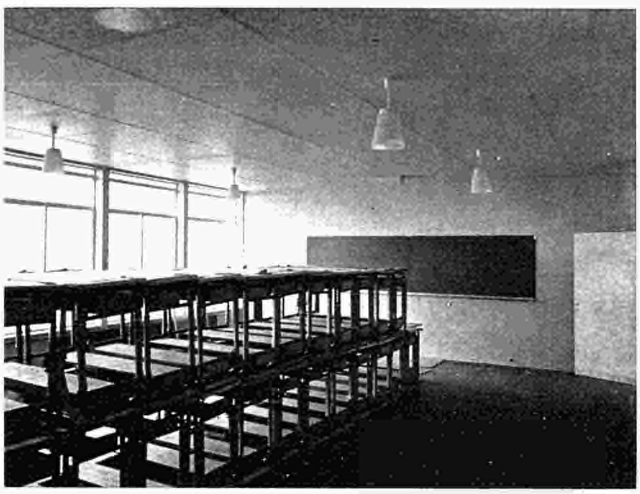

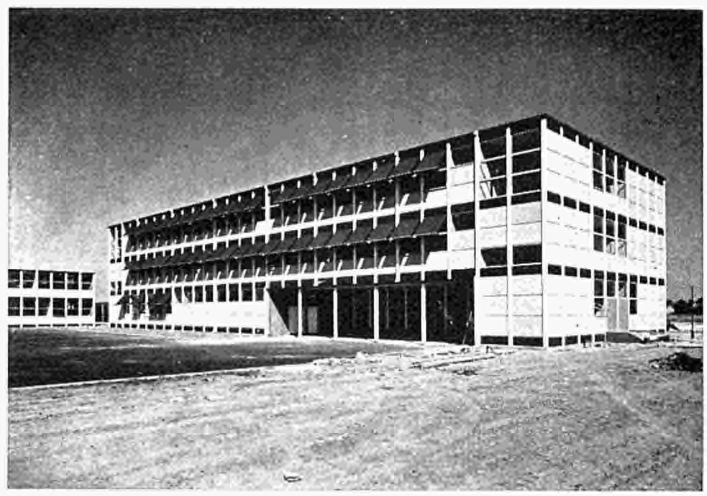

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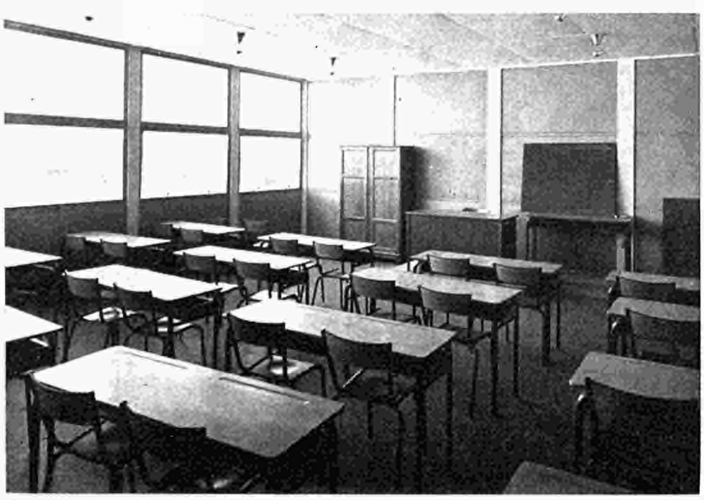

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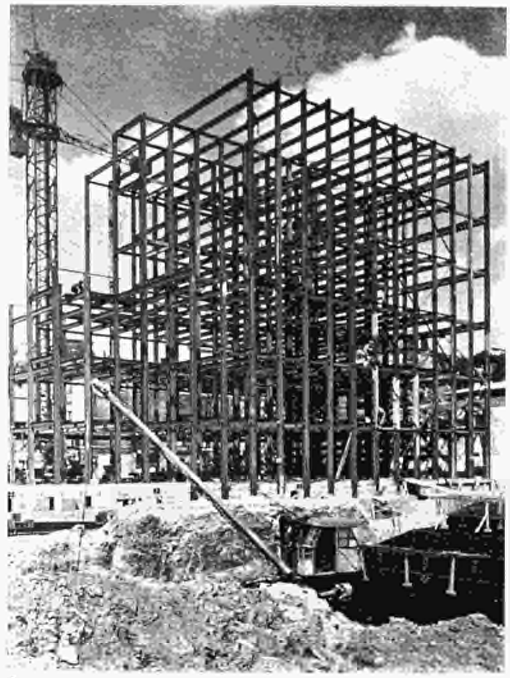

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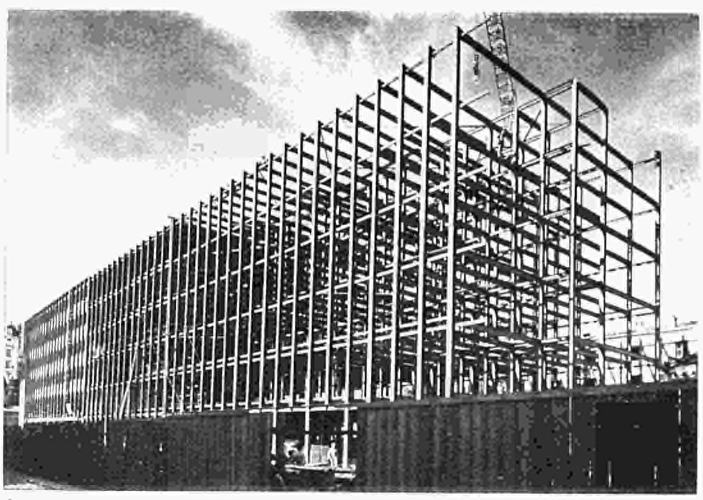

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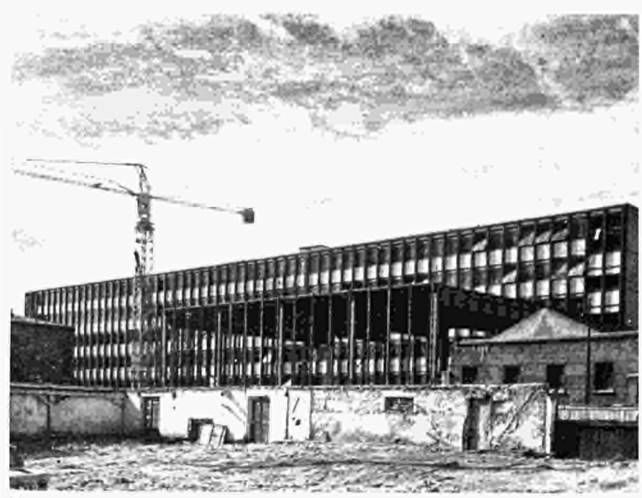

7

René MENARD

The Use of Steel in the Storage of Grain

(Original text: French)

The combine-harvester has revolutionized the harvesting of grain and its delivery to the storage agencies These processes, which used to be phased out, are now concentrated into a relatively short period.

Despite considerable capital outlay the storage agencies cannot cope with all the crops during the actual harvest, and are asking the farmers to stagger their deliveries.

Farm silos

To enable the gathering of the crops to proceed at the normal pace, farmers often provide themselves with silos for the temporary storage of the grain as the combine-harvester brings it in.

Farm silos consist of a battery of steel cells, the sides of which are made of galvanized corrugated steel sheets or plain or ribbed sheets, bolted together, or of fine reinforced wire mesh, providing circular, polygonal or rectangular containers, in which the grain is stored. The sections are mass-produced in factories and erected either by the manufacturers or their distributors, or by the farmers themselves.

Various kinds of ancillary steel fittings and equipment are required to dry and handle the grain, such as winnowing machines, worm-screws, elevators, pneumatic conveyors, driers, ventilators, air ducts, etc.

All these items taken together represent a not inconsiderable outlet, when account is taken of the ever-increasing size of the harvest. It is estimated that reasonably efficient silo capacity on farms amounts to between 1½ and 2 million tons.

Silos for collection, carry-over and transit

The marketing of grain involves a number of operations :

— silos are divided into several categories dependant on the use to which they are put;

— a distinction usually being made between types of silos for collection, carry-over and transit.

The first, which are erected in the production areas, are intended for use as depots where the grain is delivered by the farmers and graded into lots of similar quality.

When market conditions are suitable, the agencies owning the silos arrange for the sale of the grain.

The silos generally have capacities ranging from 1,500 to 8,000 tons, but sometimes the capacity may reach 10,000 tons. Following the extensions to certain silos, however, it is probable that a large number will exceed a capacity of 10,000 tons in the future.

They are fitted according to their function with suitable handling equipment which ensures that the rate of filling exceeds that of emptying the silos. Storage is carried out in low silos or in tower silos, according to the drying technique adopted.

The former consist of a series of cells of circular or rectangular section and of varying capacity, but the height never exceeds 20 feet. These are generally small silos, the capacity of which rarely exceeds 3,000 or 4,000 tons.

These silos take up a comparitively large area. By contrast, tower silos allow large amounts of grain to be stored on a smaller area. They are made up of circular or polygonal cells of relatively small section, the height of which is always greater than 20 feet and may reach 60 ft.

The continuous increase in the production of grain has induced the storage agencies to set up stocks for carrying over from one season to another. The appropriate silos are similar to those already described but have naturally to be much larger in capacity.

Finally, the increase in European and international trade in grain has resulted in the erection of large transit silos at inland- or sea-ports, the special feature of which is their extensive handling equipment, with working capacities of 500 tons per hour and more, for loading as well as discharging; all weighing is carried out at the same rate, the object being to avoid holding up barges or sea-going vessels.

The number of silos of these various kinds needing to be erected, equipped or reconstructed represents quite a large outlet for steel. The steel fabricators specializing in silos are keeping well abreast of the demand from French users.

Of the 7 million tons of storage capacity owned by the agencies, 1½ million is of steel, representing about 20 per cent of the silos at present erected. The weight of steel used depends on the type and size of the silos but is of the order of 40 to 75 lb. per ton of grain stored.

Gérard PONS

The Farm Buildings Industry

(Original text: French)

The types of buildings required by farmers have shown considerable change during the last fifty years in consequence of the widespread development of new methods of cultivation and stock-breeding.

The buildings, once so solidly built by traditional methods, have gradually given way to structures which can be erected anywhere for such purposes as stalling, the storage of materials, crops, feeding-stuffs, etc., and subsequently taken down and used elsewhere.

The past ten years have seen, particularly for stock-breeding, a demand for more specialized buildings, such as open stalling, poultry houses and commercial piggeries. To satisfy this demand, the French structural steel fabricators have put on the market a wide range of steel sheds with spans ranging from 25 to 80 ft., bays of between 13 and 20 ft. and minimum clear height between 8 and 21 ft., according to the customers' requirements.

The area of sheds in truss construction varies from 1000 to 4000 sq.ft., depending on the specific purpose, the most numerous orders being for areas of 1500 to 2500 sq.ft. Farm buildings therefore absorb a relatively small amount of steel per unit. It also happens that the units built are widely dispersed, a fact which poses certain erection problems which the fabricators must solve.

However, the requirements of farmers almost always warrant the erection of structures covering a rectangular area and the total demand is significant. To give an idea of the position, it may be recalled that it is necessary to provide about 80 sq.ft. of cover per acre of farm, which means that the area of farm buildings in truss construction in France is about 150,000 acres. These buildings are often old and unsuitable. One can therefore see the importance of this outlet, to which may be added buildings required for storing, processing and selling farm products.

After the 1914/1918 war, timber buildings met the greater part of the market. In order to render steelwork more competitive, industrialists, from 1925 onwards stored prefabricated sheds for sale in the course of the summer, the demand then being concentrated into this season of the year. This development of industrialization was possible because, on the one hand, the requirements of the farmers were fairly well defined and for a number of types only, while, on the other hand, the properties of steel lent themselves to this work.

Consequently, after World War Two structural engineers who studied the problem closely realized that the best results could only be achieved by industrializing their fabrication, i.e. by mass-production. This gave them the chance to study standard buildings, making the best use of the characteristics of the material and of the sections used. Furthermore, this allowed them to prepare designs lending themselves most readily to repetitive members which could be used in trusses, as purlins and columns, and ultimately to produce larger elements meeting special needs, such as sheds with lean-to additions of various sizes, asymmetrical sheds, etc., but always of such a kind that when erected met construction standards. Nowadays, practically all farm buildings in steel are designed in this manner.

This necessitates a work schedule designed to save time (reduction of drawing-office work, better planning, with consequent reduction in stocks of steel, and often economy of material.) However, mass-production does entail the maintenance of stocks. Although the demand today is more spread out than formerly, requirements for structural stocks are no less, and in February they can represent at least 40 per cent of the total output of an average concern.

Nevertheless, mass-production is still profitable, despite the amount of money wrapped up in stocks. As evidence, it will suffice to mention the spectacular sales achieved by the fabricators, who are steadily increasing their share of the market to the detriment of other materials. In fact, the output of agricultural buildings has increased from 13,800 tons in 1955 to 43,000 in 1963. In 1964, the figure will probably exceed 45,000.

What rolled sections are used in the construction of agricultural buildings ? Usually, angles and joists.

Trusses are almost always fabricated from angles. Purlins and columns can be made in lattice-work, but they are mostly constructed from joists.

It should be mentioned that constructional engineers who employ galvanized steel tubes now fabricate farm buildings almost entirely of tubular members. In spite of the advantages which these buildings offer the users, it seems that at the present time they cannot compete with traditional steel sheds when spans are less than 50 ft.

One tubular steel fabricator, exploiting the opportunities afforded by industrialization, now offers package buildings which farmers can erect themselves. These packages comprise triangulated components 13 and 17 ft. in length which, when assembled, provide buildings with a free span of 26 or 34 ft. and a height of 13 or 17 ft., one or other of the triangulated members serving as a column.

As regards sheeting and cladding, it should be noted that although galvanized steel sheeting, by reason of its numerous advantages is widely used, there are other materials which are competitive with it.

The weight of steel employed per sq.ft. at present in the framework of farm buildings averages 3 lb. with a covering of galvanized steel sheeting.

In conclusion, it can be reported that, in order to exploit the experience already acquired with farm buildings, fabricators are now mass-producing industrial buildings. Although the problems in this market are different and certainly more difficult to define, if the general principles of industrialization are applied to them, it will no doubt be possible to solve these problems. Recent developments in mass-production, at any rate, suggest that this is so.

Jean-Emile COMPÈRE

Rural Building in France, Residential Building in Particular

(Original text: French)

I should like to say a few words about rural building in France, particularly for residential building.

It may seem surprising to find rural building with industrialization, since such a large proportion of the rural working population are craftsmen. But it is becoming a matter of urgency to associate them and integrate them with the process as far as possible, for the problems which are now leading to industrialization are found also in the countryside — manpower, costs, a high level of demand and inadequate traditional techniques. Now rural building is notable particularly for the fact that it is very dispersed, both geographically and in time (as has been pointed out with regard to farm premises). I would emphasize that I am referring to *rural* aspect (and not just the purely agricultural), and a special feature of which is that it is individual in character. It may therefore seem curious to refer to industrialization in this connection, but nonetheless there are openings for some appropriate programmes; this is my point. Demand is very heavy : we need something like 100,000 new dwellings a year, plus renovations. The renovation of rural housing and installations calls for "major programmes of minor works". What is needed is concentration in time (say 1-2 years) and space (say within a radius of 20-30 kilometres), and planning within that concentration. This is absolutely indispensable if the demand is to be fully met, and it is here that the use of industrial methods is feasible and helpful.

The importance of prefabricating lightweight components must be stressed in view of the transport and handling problems, which are not the same as those in urban building. I recall what was said on the question of the combination of different components that could both allow sufficiently long production runs and preserve the necessary flexbiliity of utilization : this must fit as neatly as possible into rural building. I do not know whether one day we shall have series designed specifically for the purpose of rural housing: I doubt whether this is likely, or desirable either from the manufacturers' point of view. All the same, the standard components which will gradually establish themselves should meet specifically rural requirements. Not that there are so very many of these, for after all the differences are not so great between the way people live in towns and in the country.

Jean-Pierre VOUGA

Modular Co-ordination, Fundamental Research, Joint Education

(Original text: French)

Standardization, the use of models, and prefabrication are all in progress, and steel is playing a prominent part. Architects may not see entirely eye to eye, but they are prepared to accept the resulting restrictions on their creative freedom so long as there is give and take between them and those on the operational side of building.

Secondly, I feel that not enough has been said about the essential element which Dr. Henn mentioned, the need for co-ordinated sizing. This is a matter which has been frequently vented, and I feel it is worth recalling the importance of the modular co-ordination devised by the European Productivity Agency, and taken up by the International Modular Group (one of the working parties of the International Council for Building Research), to which a tremendous amount of study has been devoted. I feel it is one of our main tasks to make every effort to get this modular co-ordination of sizes adopted.

Then there is another aspect, research. You are already aware of the many difficulties which result from the illogicalities and the mistakes in certain statistical calculations and certain national and local regulations. It is imperative that laboratories for fundamental and applied research should be established and regular liaison arrangements and exchanges of material instituted between them, so as to combat the ossification of some Government and local regulations which impose absolutely preposterous fire-resistance coefficients and cause many tons of steel to be used every year to no purpose at all. Corrosion and corrosion-proofing have not up to now been sufficiently thoroughly explored for steel to be used with maximum effectiveness; it is high time they were.

As regards teaching, in the architectural and civil engineering colleges and in technical training establishments generally, it is absolutely vital to accustom students from different educational backgrounds and different branches to work together from college onwards, in order to prepare them for future activity as teams, since only by team work can we tackle the very difficult and complex tasks ahead of us.

F. CANAC

Soundproofing in Steel Buildings

(Original text French)

I should like to emphasize the problem of soundproofing steel buildings. One of the complaints of people who have to live or work in these buildings is the wearying effect of noise, the way they carry proximate and distant sounds.

I do not agree with the optimists who try and tell us that the problem has been disposed of — far from it. All the same, the physicists and acoustic experts have done a great deal of work on the subject, and definite, practical progress has been made.

E.C.S.C. comprises two sectors, coal and steel. On the coal side, a good deal of research has been devoted in recent years to the problem of noise both at the surface and underground. There has been some useful consultation between the manufacturers of the equipment and its users in a committee representing them both. I feel that a similar committee or working party might be set up within the steel consumers' group to go into these matters : it could be tremendously helpful, to the occupants of the buildings who would not have to complain, to the architects who would not be criticized, and to the steel men who would certainly find it easier to market their products.

Jan SITTIG

Planning and Human Purposes

(Original text: Dutch)

I have been impressed both by many things said in the discussion, and by much that has been left unsaid. We have seen a great many examples of attractive structures both of steel, and of steel in conjunction with other materials. We have seen that an enormous amount of labour, time, energy and money. is spent in design. Personally, I would have liked to have seen even a fraction of that energy devoted to planning, and because it is not, I am rather afraid that, in our efforts to achieve industrialization, we are rather tending to put the cart before the horse. Perhaps that explains why a number of speakers have emphasized the difficulties they come up against, as industrialization of building is all very well, but the environment is wrong, the climate is unsuitable, people are unco-operative, authorities are difficult, regulations are inappropriate, workmen don't want it, sub-contractors and main contractors don't do their job properly. I feel that many of the difficulties attaching to industrialization have arisen simply because we have started at verse two, the design, and left out verse one, planning.

That is a general comment. More specifically, I should like to thank Mr. Volbeda for what I consider his most excellent and pertinent observations. He approached the question of tolerances and fittings from the practical as well as the theoretical angle; he instanced a case where thanks to sound theory and sound planning the practical outcome was a success, and he emphasized that industrialization as a whole was what I might call, with apologies to Johann Sebastian Bach, „*die Kunst der Fuge*" (= a- „the art of the fugue" (in music), b- „the art of jointing" (in building)).

Mr. West informed us that tolerances in rolled steel products are much too great, or rather, not the tolerances, but the inaccuracy. If that is so, Mr. Volbeda has already supplied the answer there too: the upper left-hand portion of his drawing shows what the manufacturer of materials and components, including steel components, sections and so on, ought to do, namely ensure precision in his product by concentrating on quality.

Finally, may I reply to the first speaker in the discussion, Mr. Scimemi. His point took the words out of my mouth: he reminded us that we must focus on the human element. What he said really boils down to what is currently known as "human engineering," or perhaps more scientifically as "ergonomics": considered functionally, the structure — house, office, station, stairway — is there to serve not only, nor even primarily, a technical, but a human purpose, for it forms the background against which people work, live, or travel.Mr. Scimemi also referred to the great difficulty of focusing on the human element — that human beings vary so enormously. He showed us a graph, a bell-shaped curve indicating, for example, human stature, and said in designing one should take as ones point of departure the *average* person. In this one instance I find myself unable to agree! As a statistician, I know what an average is, and I would warn you against letting this kind of enthusiasm for the "average person" run away with you. To take an arbitrary example: suppose one day we decide to standardize the height of doors. The people who will have to go through the doors, according to this bell-shaped curve, will be from 5 ft. to 6 ft. 6 in. tall. If we design doors for the average person measuring 5 ft. 9 in. (whom you can call the Le Corbusier module, or anything else you like), that will simply mean that half the people are going to bash their heads.

To put it in the terms I was using this morning, the degree of inconvenience is asymmetric: if the door is too high, the inconvenience is minimal, but if the door is too low it is very considerable. The solution is precisely what I was trying to indicate mathematically this morning. Industrialization ought to mean that one designs the door not for the average person, but, say, for somebody who is percentage-wise pretty nearly the tallest person of all.

However, this does not alter the fact that I am most grateful to Mr. Scimemi for centring the discussion once more on the human element, for, notwithstanding mathematics and technology (concerning steel or anything else), and architecture, I believe our whole function is to help to serve human purposes better.

Findings

Standardization of components, standardization of types and prefabrication are steadily progressing, and they alone can fulfil the tremendous requirements of our time.

In this sphere, steel construction undeniably plays a part of prime importance.

In the first place, there is the urgent need for standardization of structural components, at least on a European scale. The system of modular co-ordination of dimensions recommended as far back as 1953 by the *Union Internationale des Architectes* (International Union of Architects), elaborated by the *Agence Européenne de Productivité* (European Productivity Agency) and continued by the International Modular Group, which has since become the *Commission de Travail du Conseil International du Bâtiment* (Working Committee of the International Building Council) (C.I.B.), should serve as the basis for such standardization, which should in any case comprise a definition and lay down limits for the tolerances.

Those industries which already have their own standardized products should bring the further development thereof within the framework of this European standardization.

In order to achieve the closest possible co-operation between the iron and steel industry and the construction industry, it would, secondly, be advisable to promote organizations for basic and for applied research. The architect, the iron and steel metallurgist and the manufacturer should be brought into the picture already at the design stage of the components and while constructing the equipment and when initiating the sequence of manufacturing operations.

This research should more particularly relate to the grades, sizes and cross-sectional steel shapes best suited for the purpose, and to the functional, aesthetic and economical design of these members.

Thirdly, the Community's activities should be concerned with:

— drawing up a schedule of the most urgent needs of each country with regard to urban structures, residential buildings, schools and hospitals, so as to enable the most efficient possible use to be made of steel;

— revision of the national and local regulations relating to safety, to corrosion protection, and to administrative and financial procedure; this revision is indispensable if present-day manufacturing techniques are to achieve their full efficiency;

— revision of the rules and practices that are currently operative with regard to the placing of orders and the awarding of contracts;

— better definition of the services and the responsibilities of the partners towards one another.

Fourthly, it appears essential to introduce into the professional training of architects and civil engineers, and also into that of technicians and artisans, a body of theoretical and practical instruction directed towards industrialized constructional techniques in steel, in order to create conditions favourable to co-operation between men with any kind of professional or vocational training.

Fifthly, in order to co-ordinate all these activities, it is proposed that a European institute for research on industrialized architecture be established which, jointly with existing organizations, would have the task of promoting a high-class architecture, suited to the needs of man and to the resources of our time, and also of putting it into practice through the efforts of new design and construction teams consisting of architects, engineers, manufacturers, artists and sociologists.

This institute would, at the same time, strive to promote, in the various regions of the Community, working organizations embodying the services of highly qualified persons from the member countries and from other countries also.

It would be advisable to agree with the High Authority a time schedule for implementing the various points of this resolution.

WORKING PARTY VI :

New Methods Employed in the Preparation of Building Plans and in the Calculation of Steel Constructions

Chairman :

Dr.-Ing.Walter PELIKAN

Rapporteurs :

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Henri LOUIS

The Working Party dealt with new methods of preparing building plans and calculating steel constructions; special attention was given to questions concerning the theory of elasticity and its application in building steel superstructures, and also to problems in connection with steel bridges.

The rapporteurs and a succession of other speakers discussed the question of how the latest advances in theoretical knowledge could be exploited and fabrication rationally organized to ensure the most economic possible results. To prepare the way for the practical application of recent research findings, it was felt to be necessary that the regulations relating to the calculation of steel constructions should be amended in line with these findings at the earliest possible date. In addition, many old-established but by now obsolete methods of calculation were still in use and were leading to uneconomic planning; a drive needed to be made to disseminate information on new research results and knowhow.

The application of these advances in connection with design was producing new constructional forms (*e.g.* shell structures made of pre-stressed wire ropes) which if carefully planned could be quite economic, and suggested new openings for the use of steel in fields hitherto regarded as the preserve of reinforced and pre-stressed concrete.

As well as seeking to build as economically as possible, it was necessary to bear in mind the importance of making steel constructions pleasing in design: the erection of economic but unsightly structures was liable in the long run to have an adverse effect on the widespread adoption of a material.

To make for more and more economic building it would be necessary to carry out constant research both on the theory side and on ways and means of securing efficient fabrication; for this to be fully effective it should not be conducted separately and independently in the individual countries, but as far as possible be directed by broad-based supranational bodies.

Hermann BEER

Recent Developments in Design and Calculation of Structural Steelwork

(Original text: German)

Introduction

Steel construction has in recent years gone ahead with great vigour. This development has been seen in theory, design and actual construction alike. As rapporteur for this specialized study group I have made it my business to acquaint you with the important progress which has been made in this field and to give examples of such developments.

If one compares today's steel structures with those, say, of the period before the Second World War, it will be seen that a change has taken place in basic thought, and this can perhaps be made more clear if the following matters are considered :

(1) In place of statically determinate structures or structures indeterminate to one or two degrees, we now have structures which are statically indeterminate to many degrees (1).

(2) The provision of a special structural member for each single structural purpose has given way to the conception of a "monolithic" integrated structure suitable for all cases of loading (2). Often a combination of steel skeleton with space-enclosing elements is produced, and this adds considerably to the rigidity and load-carrying capacity of the structure (Fig. 1).

(3) In addition to structures composed of two-dimensional braced framing or plated constructions, increasing use is now being made of shell structures in lattice or stressed-skin elements (3).

(4) Connections between individual structural units are made nowadays in the shop almost exclusively by welding, which approaches nearer and nearer to perfection, whereas on site joints are made by welding or with the aid of high-strength bolts (4).

(5) Further advances in the theory and design of steel structures, especially of surface constructions, tends, in association with research (5), not only to new developments in construction itself, but also to the more economical dimensioning of structural members.

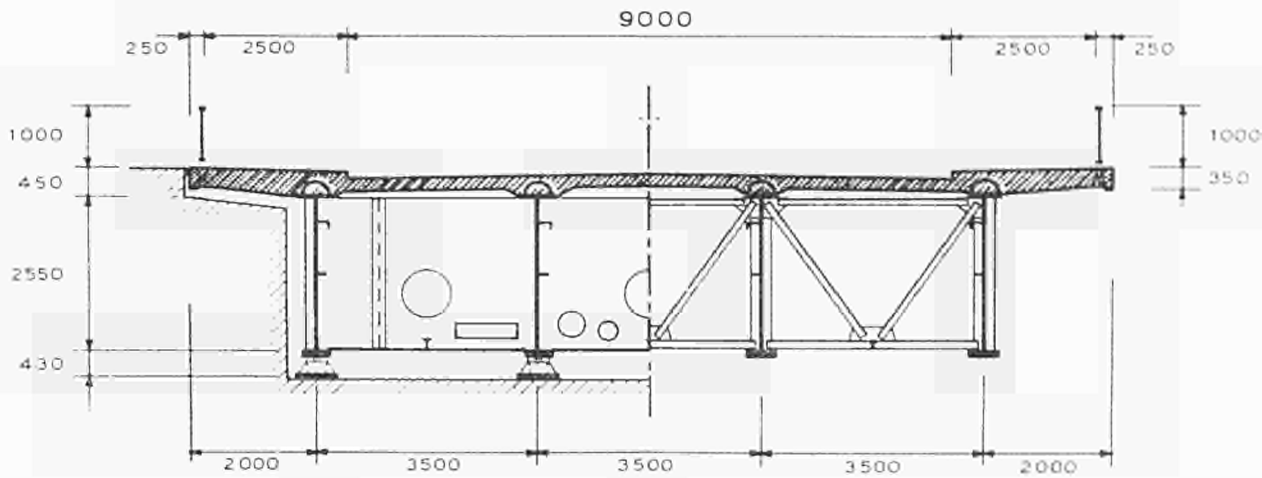


Fig. 1

(6) The idea of security against collapse and potential incapacity is undergoing a fundamental change (Figs. 2 and 3) and is leading us to adopt theories of probability and statistics.

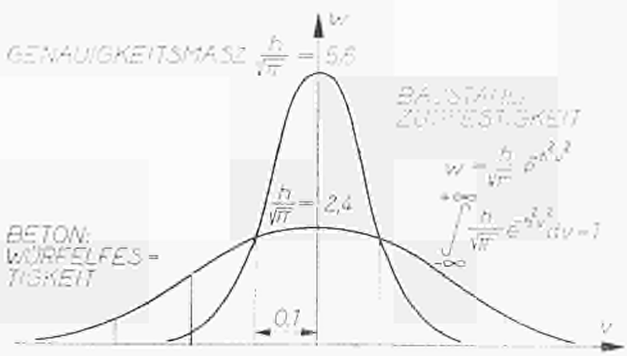


Fig. 2

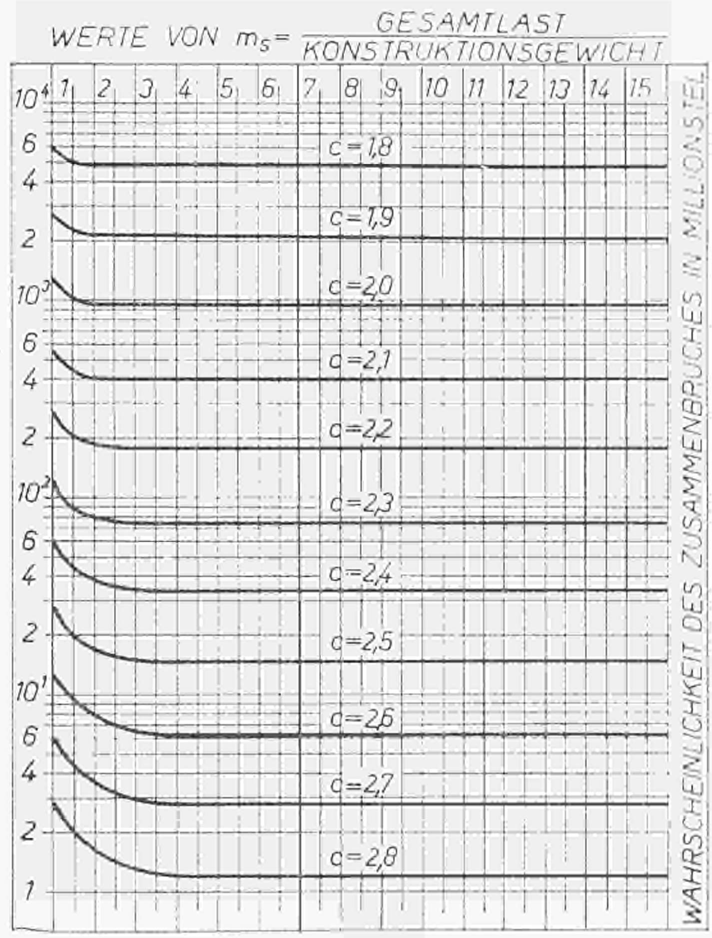


Fig. 3

Steel construction in Europe, furthermore, tends towards the utmost economy in steel tonnage, but this must be accompanied by continuous simplification and saving of cost of shop work and of erection work, if higher labour costs are not to exceed lower material costs. The very fierce competitive struggle with reinforced concrete construction and even, in recent years, with timber construction for normal building types impels the designer of steelwork and the structural engineer to obtain maximum performance from a given quantity of steel.

In what follows, some ideas will be suggested which point out the way for theory and design in modern steel construction. First we propose to deal with general theories—those which cover the broad field of steel construction and then to introduce characteristic examples from different types of steel construction.

Structures in thin metal

By "light-weight buildings" we mean generally structures composed of thin-walled elements of efficient cross-section. The load-bearing structure of light-weight buildings consists in the main of braced frames whose connections are welded or perhaps, in these modern days, occasionally made with adhesives (6)

(Fig. 4). Such load-carrying structures can be built of tubular sections, wire ropes or steel rod as well as of plates, sheets or rolled steel sections.

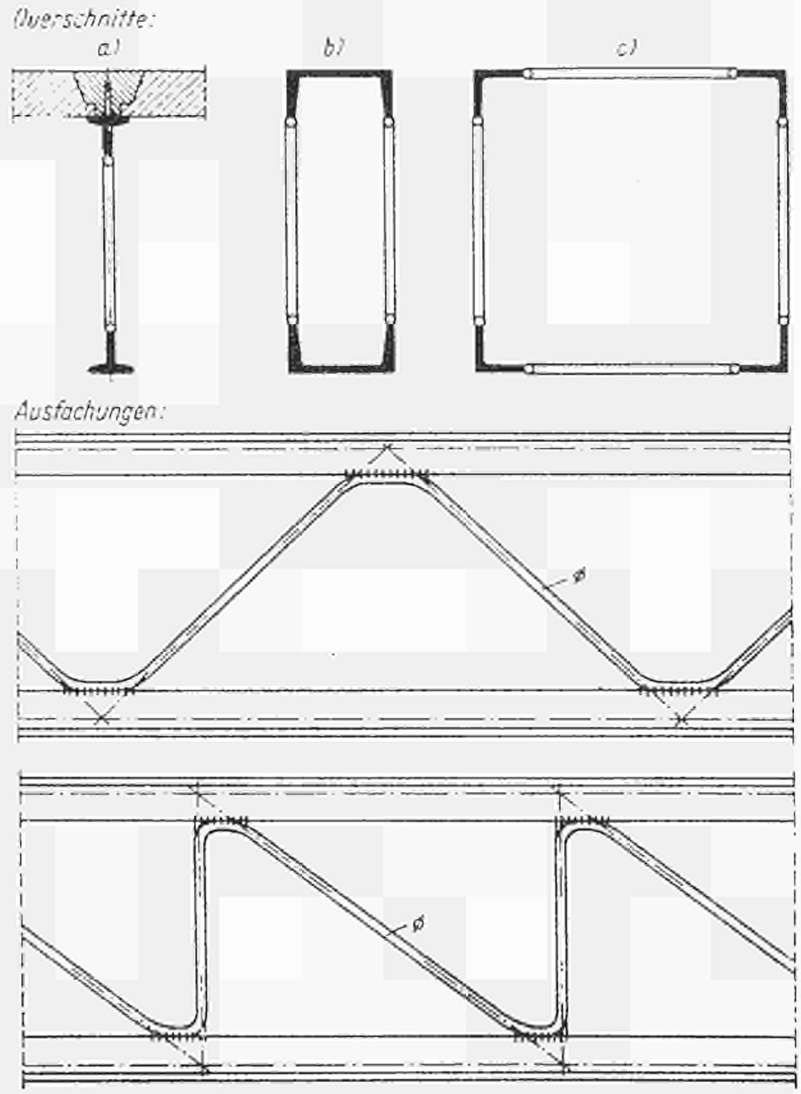
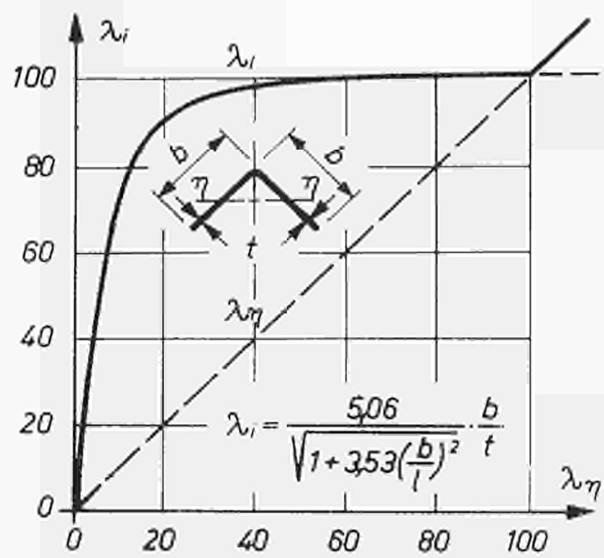


Fig. 4

Here we will deal as fully as possible with the principle of light-weight construction and will discuss all structures that show the utmost economy in steel in their construction. We include under this heading structures of great length and breadth made from thin steel plate (7) such as are used in building large bridges.

Light-weight construction for buildings

The structural frame for a single-storey hall in light-weight construction or for a steel skeleton frame consists as a rule of lattice girders, which are treated either as simply supported frames or form, with connected plated legs, a portal-frame system. The sections of the members are often in these cases open profiles. The theory of struts made of open profiles has advanced a long way in the last ten years. Yet practical conclusions are not always drawn from the theory, although the economic efficiency of any structure depends on correct dimensioning of its members. Figure 5 shows the greatly increased theoretical slenderness ratio that governs the actual dimensions of the strut and can be accepted when thin-walled open angles are used as struts with slenderness ratios below 100. The theoretical slenderness ratio for combined flexural and torsional buckling far exceeds the value λ_η which applies to flexural buckling alone. Thin-walled cylindrical tubes, however, behave exceptionally well as struts in respect to both elastic buckling and semi-plastic buckling. Below the yield point this is explained by the fact that the hollow cylindrical section of given sectional area has the highest modulus of section about any axis of its section so that it is ideal for bars that need to have the same slenderness ratio about any axis. Even under buckling in the plastic range this profile is remarkably efficient.



$$\sigma_K = \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2\left[1{,}5\,\frac{b^2}{l^2}\tau_E + 0{,}425\,\tau_G\right]$$

$$\sigma_K = \frac{\pi^2 E\,\tau_E}{\lambda_i^2} \longrightarrow \lambda_i$$

Chwalla	$\tau_G = \tau_E = \frac{T}{E}$
Bleich	$\tau_G = \sqrt{\tau_E}$
Kollbrunner	$\tau_G = \frac{1}{2}\left(\tau_E + \sqrt{\tau_E}\right)$

Fig. 5

Figure 6 shows buckling loads for various profiles of equal sectional area in relation to their effective lengths for buckling. Here, however, the wall thickness was chosen so that, within the slenderness limits that occur in practice, buckling by flexure was still critical for the open profiles. The curve of buckling loads for the tubular section stands highest of all the types of section shown. Conditions are still more favourable for the tube as wall thicknesses are reduced, for then flexural and torsional buckling becomes the determining factor for open profiles and the curves for buckling loads fall away sharply (Fig. 7).

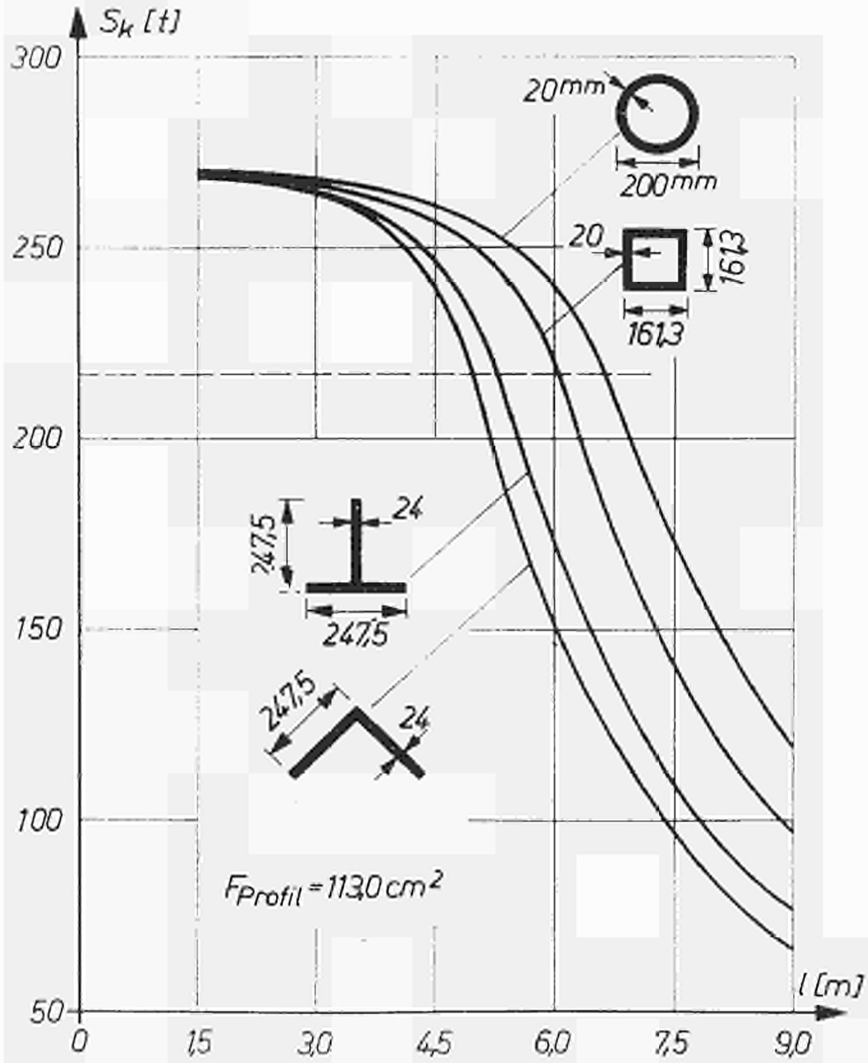


Fig. 6

If we consider local bulging of the walls of the tube, here again the cylindrical section has the advantage, for these bulges are critical only with very low ratios of thickness to diameter (t/d). Investigations have shown that even with d/t = 100 local bulging of the tube governs the dimensions only when the slenderness ratio is less than 30. K. Klöppel and W. Goder have plotted new theoretical buckling stress curves for tubes and the accuracy of these has been confirmed by tests (Fig. 8). Comparison of these values for buckling stress with those given by German Standard DIN 4114 clearly shows the advantages to be gained by using tubes as bars in compression.

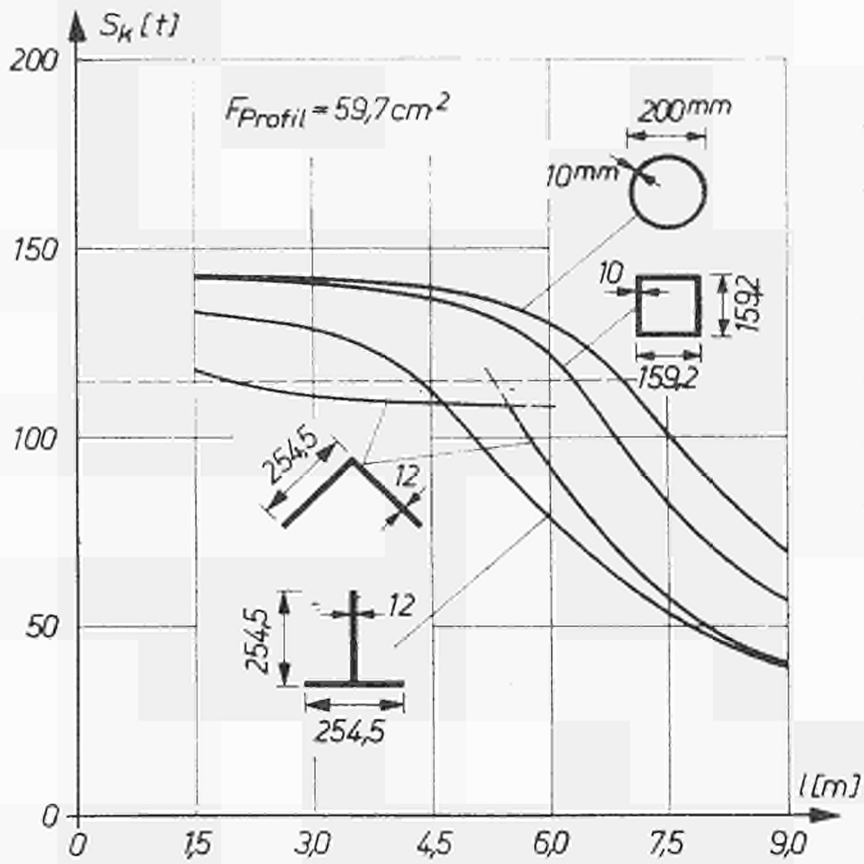


Fig. 7

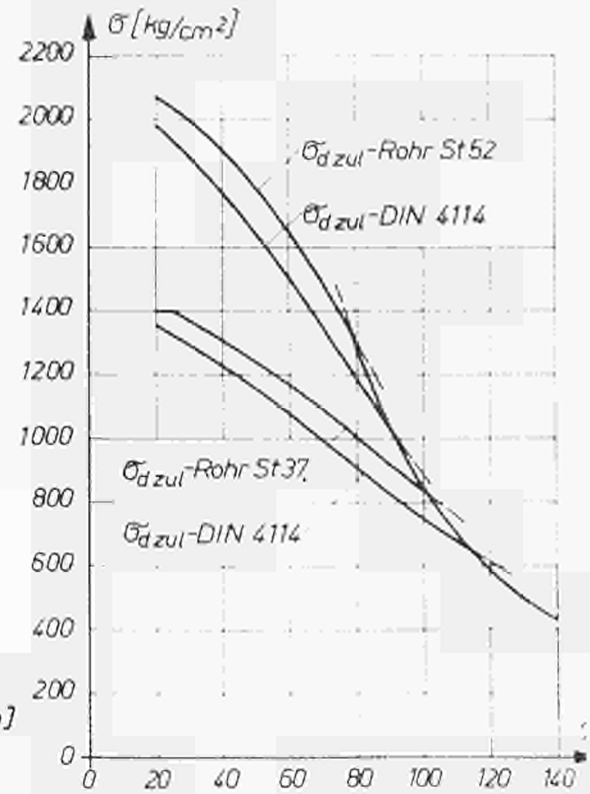


Fig. 8

In constructional practice, the fitted joints of tubes can easily be welded to form latticework (Fig. 9). Here the newly developed flame-cutting machine, which gives the correct settings automatically, is a great aid to economy in fabrication. The use of gusset plates and straps has proved to be inefficient for this purpose for, especially at the ends of the tongues on the tube, a bi-axial stressing with high concentration of stress occurs.

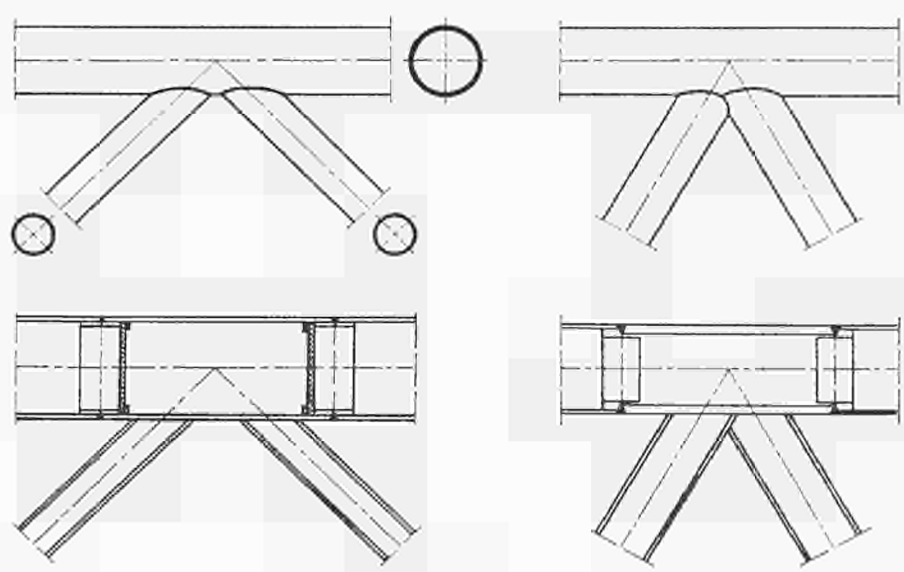


Fig. 9

In bridge building, particularly in built-up areas, steel bridges with self-spanning continuous decks (Fig. 10) in closed box-sections or lattice beam construction have made their appearance. The structural system consists of two or more webs, one upper-boom platform or diaphragm forming also the carriageway and one lower boom diaphragm or separate low-level ties. The choice of plate thickness (Fig. 11) for the individual webs and flanges vitally affects the total weight of the bridge. In order to ensure an adequate margin of safety for the steel plate against buckling under compression and shear, fairly thick plates are required in the case of bridges with high main girders unless stiffeners are fitted along the length. The structural engineer has therefore to direct his investigations toward finding the optimum relationship between metal thickness and number of stiffeners (Fig. 12). The theory of stability of perfectly flat plates, with multiple stiffeners, free of

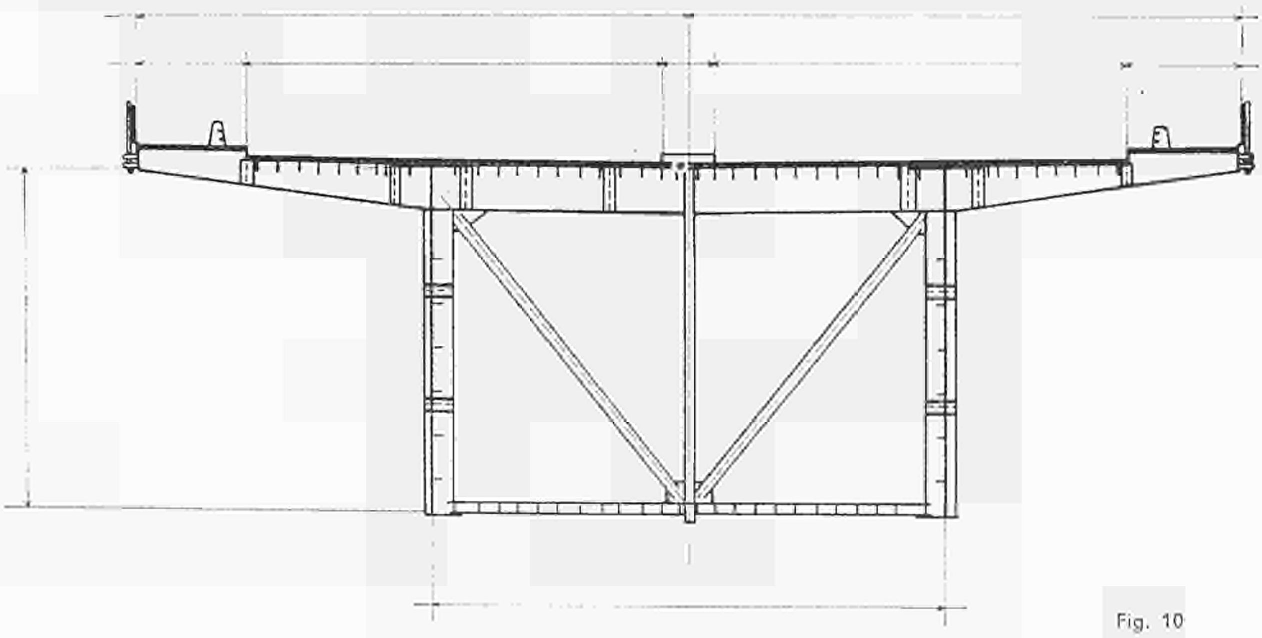


Fig. 10

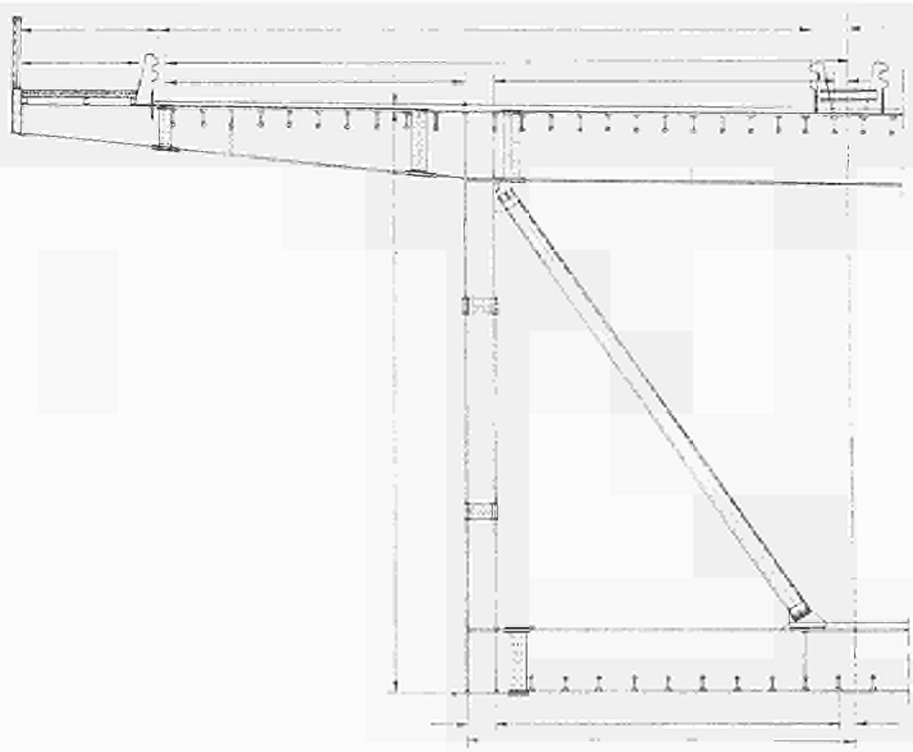


Fig. 11

residual stresses and subject to compressive and shear forces, is applicable to this case. Tables, and the availability of electronic computation of the sources of bulging have simplified the working-out of many variant forms with different combinations of stiffeners and metal thicknesses, so that today one is in a position quickly to investigate a quite high web plate having many stiffeners. Figure 12 shows how closely the total amount of steel area required varies with metal thickness and number of stiffeners. Whereas in the example given with web under transverse bending the minimum area of steel is obtained with five stiffeners, the reduction of shear force obtained by increasing the number of stifferers permits a further saving in area.

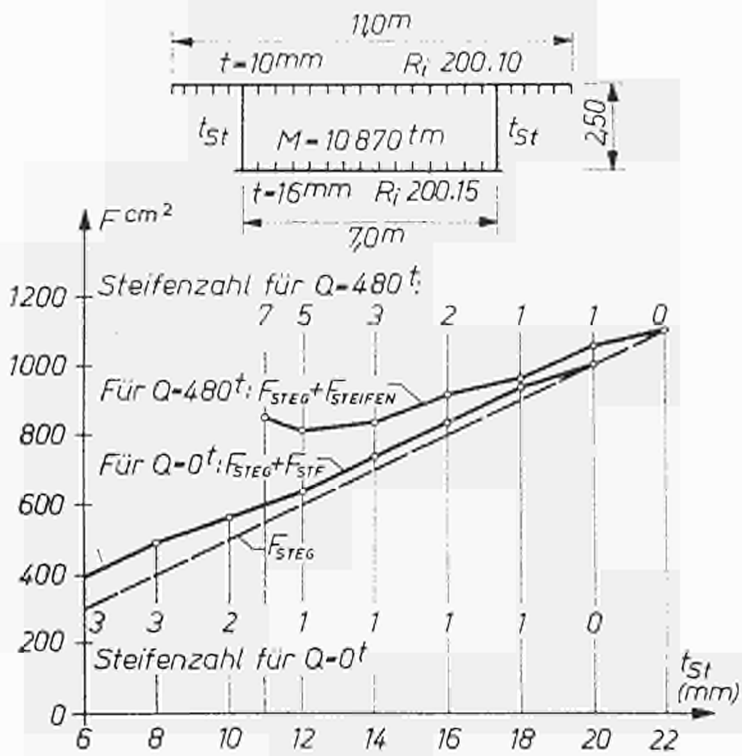


Fig. 12

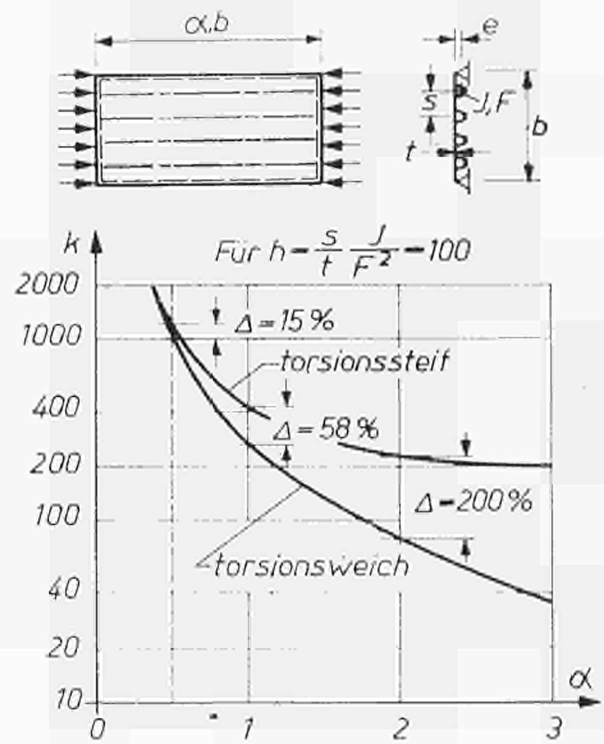


Fig. 13

The stability of the panel of plating is considerably affected by the form the stiffeners take. If they are treated as open-sided profiles and are stiff enough in bending they leave the plate free to twist about their long axes, whereas stiffeners of box section produce relatively rigid fixing of the plate edges. Figure 13 shows the percentage increase, Δ %, of the bulge factor 'k' that is obtained by fixing box sections instead of open profiles as stiffeners. This factor relates to various ratios α of panel width to height of web. In figure 14 however, bulge factor 'k' is related to the ratio ρ of sectional area of web to total sectional area. The great advantages of closed profile stiffeners over open-section stiffeners is evident when the optimum value of $\rho = 0.4$ is reached in either case. These investigations were conducted by A. Pflüger.

Although in aircraft construction the bulging of sheets is permissible even under working loads, in steel construction a corresponding margin of safety is required against plate bulging under maximum permissible loading. To fix the value of this safety factor it is important to know whether the bulge heralds failure of the parts or whether with further increase of loading the forces may perhaps transfer themselves to other points of the section. In such a case calculations need to be applied to a suitably reduced width of plate called the "contributory width" (Fig. 15).

In recent years, much research has gone into the question of plate bulging beyond the critical range and it has been established that, even in the bulged state, plates and sheets are still able to take further load (Fig. 16). If one ignores a bulge whose greatest width is equal to twice the metal thickness, then, according to

Volmir, for the plate under a direct force one can allow an increase in the critical bulging stress of about 50%. In our diagram the critical bulging stresses are given for corresponding values of bulge amplitude 'f'. One of my thesis students, H. Bergler, has carried out similar research into steel plate and sheet under shear stressing (Fig. 17). I made a report on this at the last Austrian Conference on Structural Steelwork.

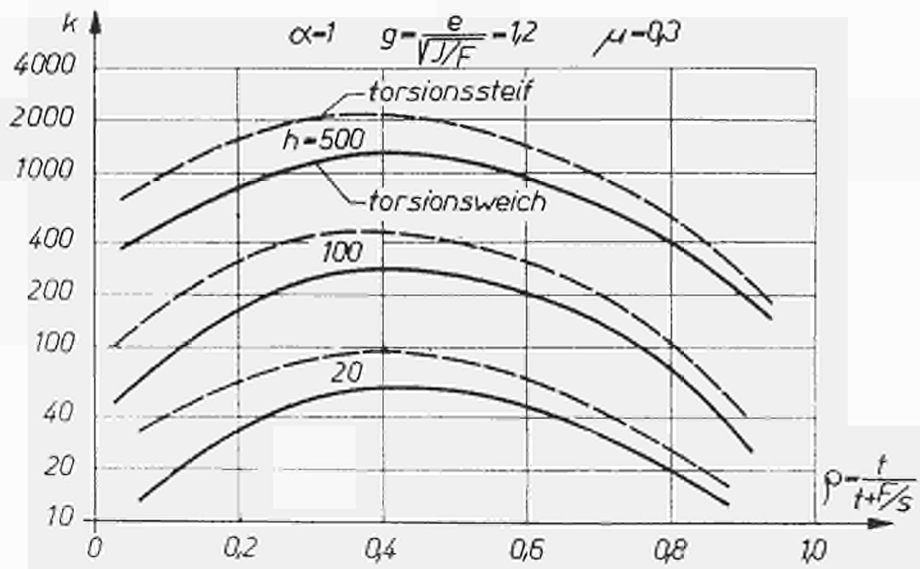


Fig. 14

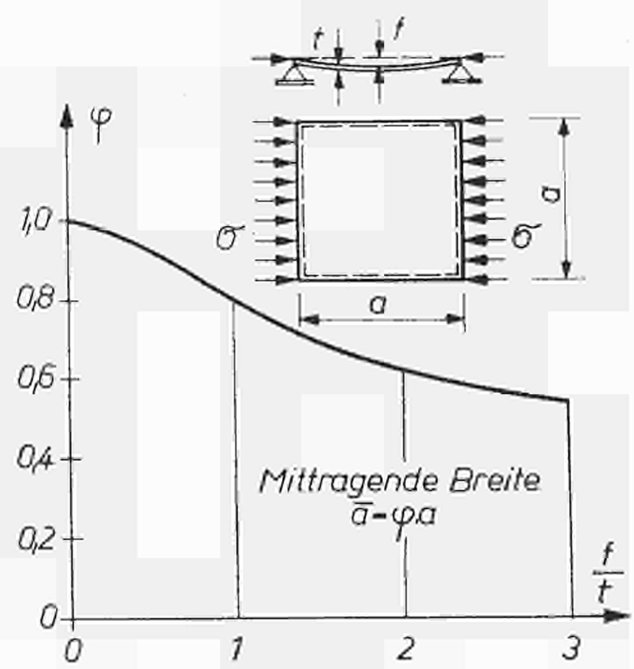


Fig. 15

While dealing with conditions beyond these critical points it is important to extend one's research to incipient bulging of sheet metal firstly because such bulges cannot in practice be entirely eliminated, and secondly because with systematic use of slightly precambered sheets higher bearing strengths can be attained than with flat sheets. This is, for example, the case when the formation of wave-like bulges, which would reduce the resistance of the sheet to failure by bulging to its lowest level, is in fact made more difficult. In figure 18 an incipient symmetrical bulge is compared with a trifling asymmetrical wave-like bulge in a flat panel.

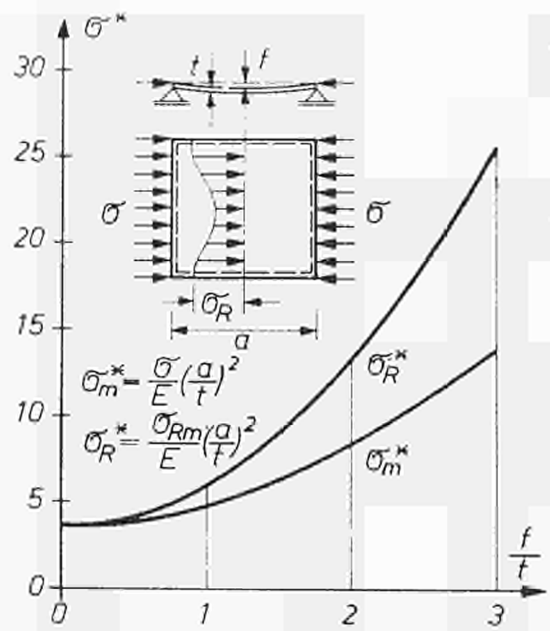


Fig. 16

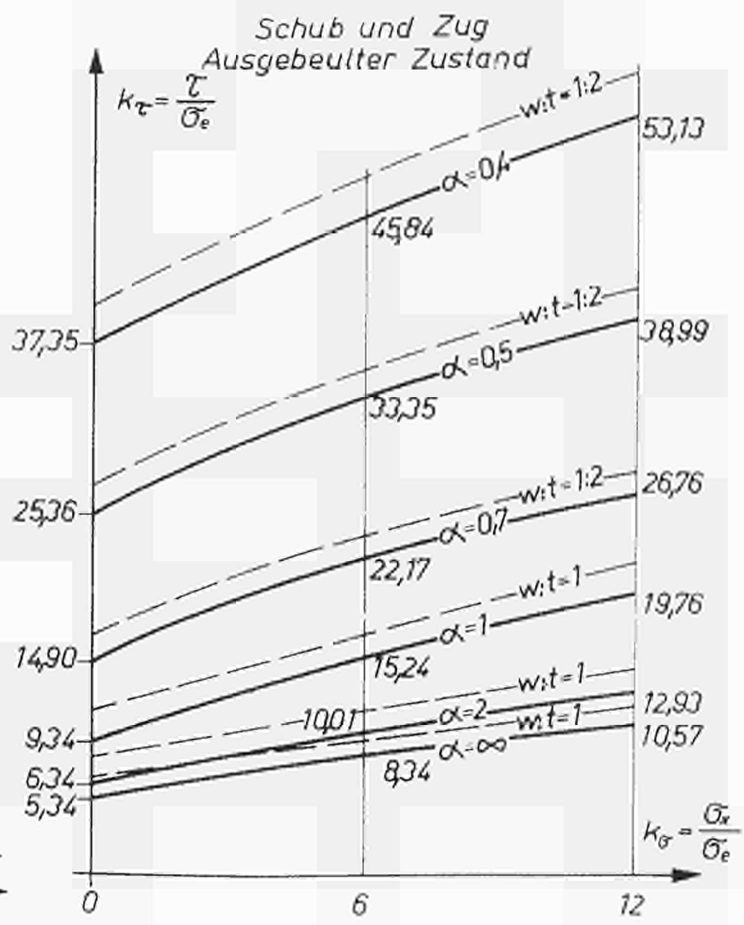


Fig. 17

With increased loading the latter bulge breaks through or the other bulge continues to grow. In conclusion let us examine briefly the question of how forces are transmitted into wide-flange plates (Fig. 19). The diminution of stressing, depicted here in the top flange, above the central support of a continuous beam suggests that, especially at the points where the forces are applied, the contributory width of the plate must, according to the elastic theory, be drastically reduced. By adopting the collapse load method, however, it can be

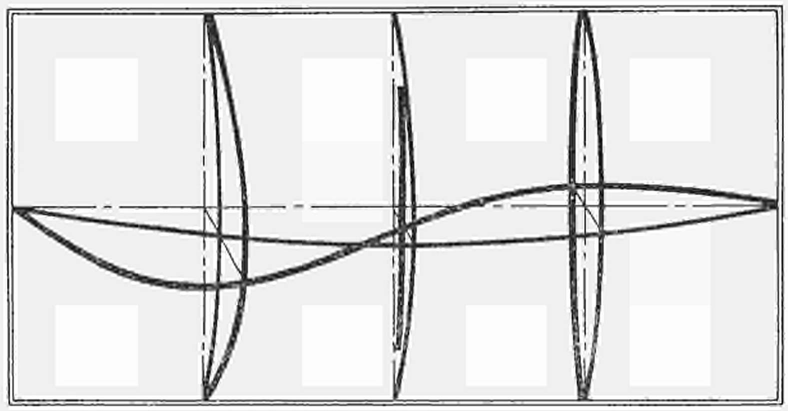


Fig. 18

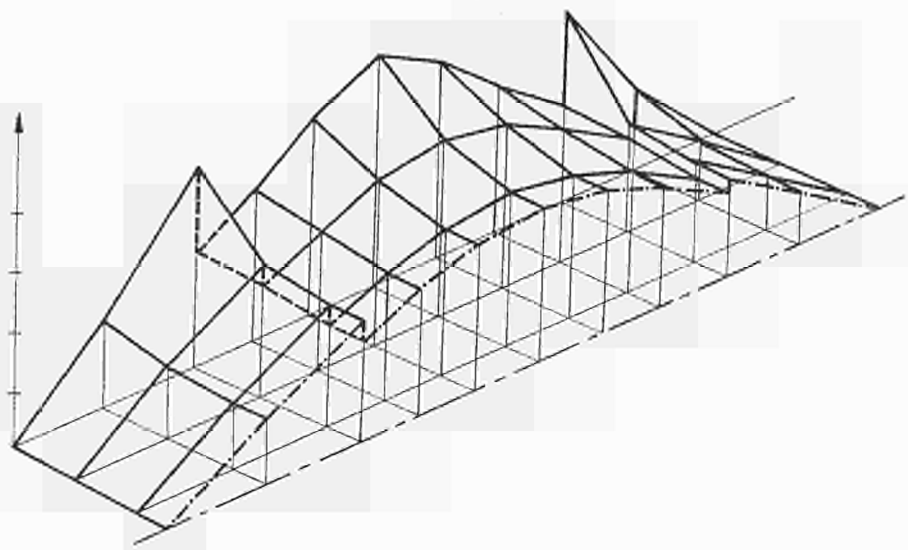


Fig. 19

shown that a local flow of metal brings about a redistribution of stresses, which eventually at the point of collapse gives an even distribution of stressing over the plate. The load factor is therefore not adversely affected by any uneven distribution of stresses in the flange plates. Nevertheless, in calculations, and in choosing a factor of safety, one has to watch for any possibility of recurrent or fluctuating loads, for in such cases metal fatigue and cold-straining due to repeated overstepping of the elastic limit have to be reckoned with.

Wide-span roof structures

New and remarkable forms of construction have been evolved in recent years for the structures of wide-span single-storey buildings. Apart from the use of wide-span lattice girders of tubular construction, roof structures of shell or suspension types have been built with the object of saving as much steel as possible. In these examples, local loading and loading over the entire span of the hall are absorbed together in a "monolithic" or integrated structure, whereas in more orthodox wide-span roofs the load-bearing function is split up between roof-covering units, roof framing, purlins and roof girders.

Tubular structures

I have already mentioned the advantages to be gained by the use of steel tubing in braced frame structures. In the all-welded exhibition hall shown in photograph 8, the triangular, intersecting latticed frames are of tubular construction so that all cleats, fish-plates and gussets can be eliminated.

The Innsbruck airport building (1) again shows an arrangement of triangulated tubular girders rigidly connected to their columns to form, in combination with the purlins, also of triangular cross-section, a space-frame construction.

The theory of statically determinate space frames already finds a place in the classical text books on statics. The possibilities of electronic calculation are full of promise for the production of space-frame systems that are statically indeterminate to a high degree and are capable of taking their loading very efficiently with ample margin of safety. With tubular latticework, as my own investigations have proved, tonnage savings of about 25% can be realized as against the use of orthodox latticed construction. This saving must, however, be offset in part by the higher price per ton of tubular material.

Shell roofs

The use of lattice construction for vaulted roofs is of course not new. Among the more recent shell-type roofs, the "Ledererkuppel" (9) deserves special mention for its simple construction and for its very low consumption of steel. Prof. Lederer builds this spherical shell with three systems of tubing which form triangular networks, so enabling the shell to carry loads anywhere on its surface by skin stressing. However, the deformations of the lower tensile ring and the upper compression ring that are required to stabilize the membrane are not compatible with the deformations of the membrane itself, so that uneven localized deflections occur at both edges of the shell. The extremely thin membrane that is constituted by the domed network ensures on the other hand a very abrupt cancellation of these irregularities at the edges, so that only in the immediate vicinity of the rings are they at all significant. The resultant forces at the nodes of the triangular network (10) are taken up by a friction type of clamp. The Ledererkuppel is extremely suitable for carrying all applied loads (self-weight, snow and wind loading). By reason of its light weight and speedy erection it is very economical for roofing halls circular on plan. Thus, the steelwork for an exhibition hall of 307 ft. (93.5 m.) diameter comes to 6.76 lb. per sq. ft. (33 kg./m².), so that the Ledererkuppel easily beats any reinforced concrete dome on the score of economy and opportunity for architectural expression.

The ability of curved membranes to carry high loadings has led to the development of suspended shells in thin steel sheet which take the form of spherical or conoidal structures (Fig. 20). The wide-spanning spherical shell is well able to carry high surface loading in spite of its trifling thickness of metal, but cannot, without considerable structural rearrangement, support point loads. The conoid shell, however, is specially suited for this purpose and can be constructed in single or double form. I have already, in "Stahlbau 1963", reported on a conical shell roof with crane suspension along the axis of the conical shell. This system has in the meantime undergone further development (11), with the important improvement that the upper latticed ring beam is omitted and the heating and ventilating plant fixed along the middle part of the conoid shell (12). The load-bearing skin forms in this system the roof covering too, and a 1" (2.5 cm.) thick coat of sprayed asbestos provides adequate thermal insulation.

A further development was carried out in Austria with sports halls, and figure 21 shows a Sport and Physical Culture Centre of 203 ft. (62 m.) diameter. This has a suspended conoidal roof in sheet steel, which nowhere exceeds 5/16" (8 mm.) in thickness, and a compression ring in reinforced concrete precast units. The roof is carried on raking supports. The thrust of these is resisted by a circumferential tensioned cable in such a way that only vertical forces are carried down to the foundations (Fig. 22). The high load-carrying capacity of this shell permits even a swimming bath with terraces and bar to be built on the roof. The weight of the water and the structures on the roof serve here as pre-tensioning ballast for the membrane so that this requires no stiffening.

Roofs over halls of rectangular plan can be constructed with steel sheet of double curvature to give folded three-dimensional structures. Horizontal tension is taken up by a special adjustable tie. The double curvature provides a high degree of security against uplift by wind.

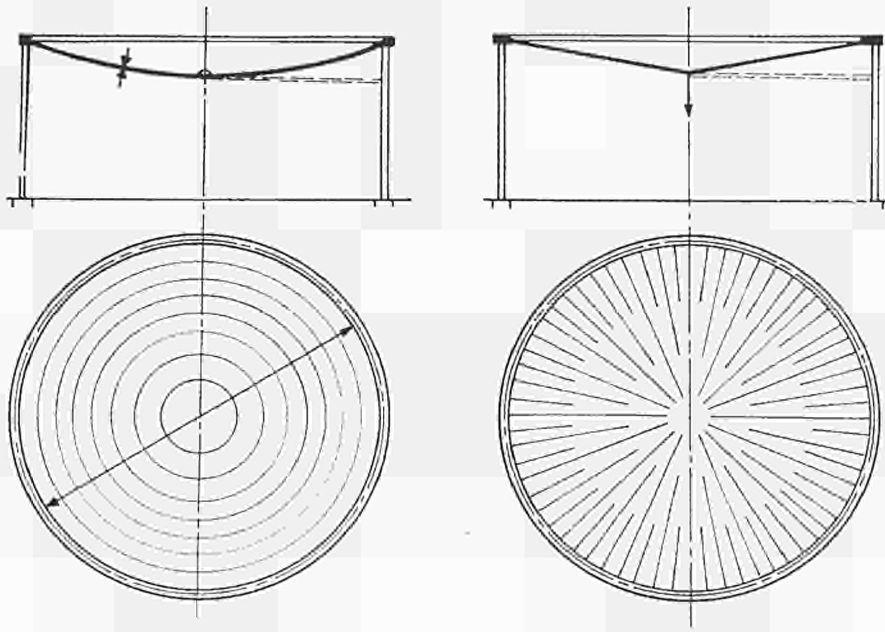


Fig. 20

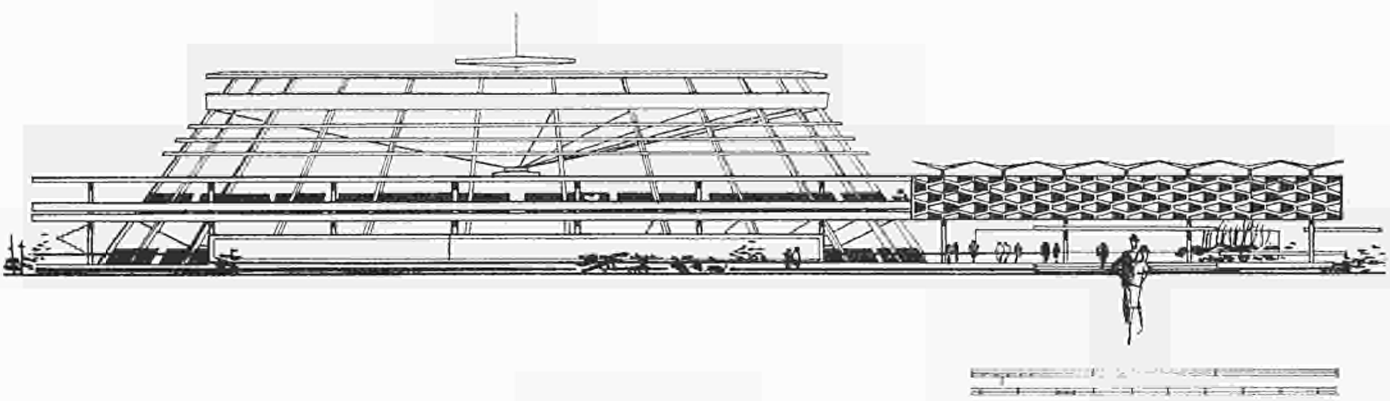


Fig. 21

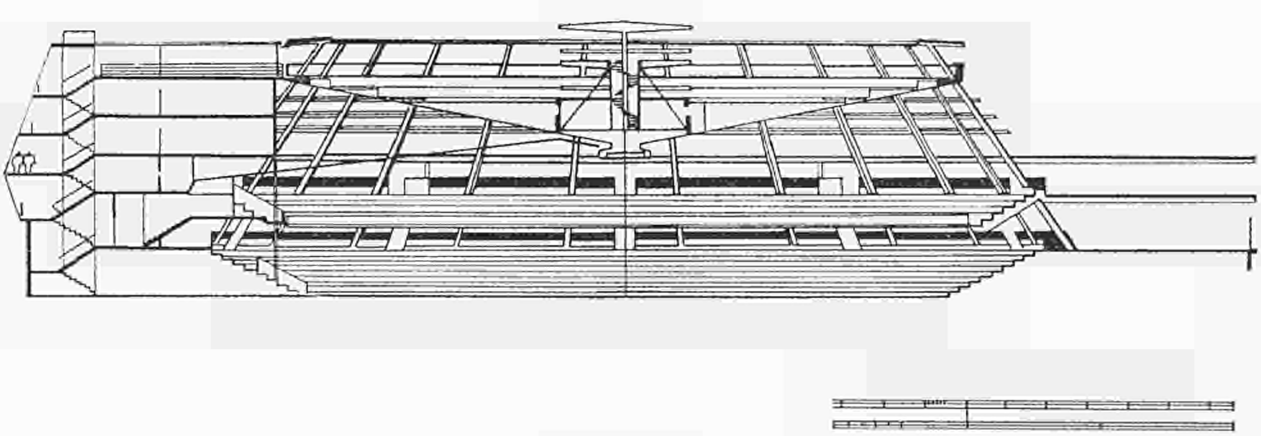


Fig. 22

The potential uses of shell structures in sheet steel for roofs over buildings of wide span are by no means exhausted. The possibility of calculating shell structures of any shape with the aid of electronic computers puts the engineer, in collaboration with the architect, within reach of a great variety of structural forms. Among these modern shell structures the conoidal and hyperbolic-paraboloid shapes deserve special mention. The construction of these from sheet steel is a relatively simple business, if only because the surfaces of such shells include a large number of straight units.

Suspension roofs

The high tensile strenght of ropes and cables made up of high-strength wire makes these exceptionally suitable for carrying tensile forces. Stretched wire rope construction is used particularly in the building of aircraft hangars, with a view to obtaining long front walls, un-obstructed by columns (Fig. 23). The vertical resultant of the tensioned cable requires, of course, an anchorage in the foundations and here the annexed buildings that are usually required provide any necessary counterweight. An even distribution of dead load is obtained by symmetrical planning of the roof (Fig. 24), so that special precautions against uplift need be introduced for unbalanced snow loading only.

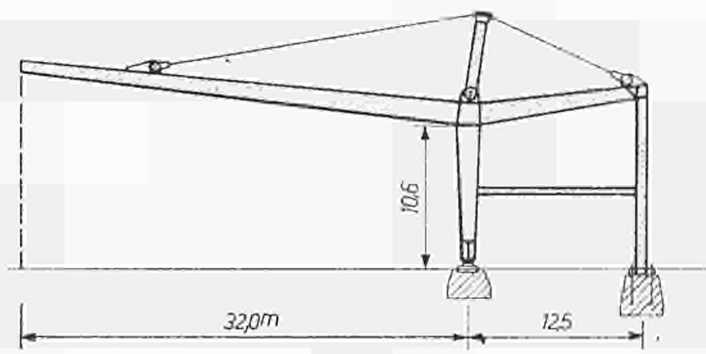


Fig. 23

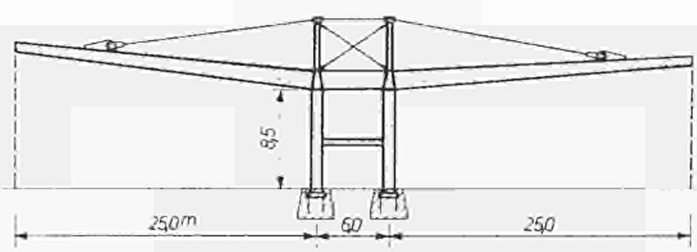


Fig. 24

Steel wire rope is always admirably suited as a structural element for use when covering in extensive areas without columns, so long as the horizontal pull can conveniently be taken by solid construction. Where the plan is circular a closed ring beam under compression or, when elliptical on plan, a hogback girder, conforming to the line of thrust, gives a suitable construction (Fig. 25). With any desired plan the pull of the ropes may be taken by elevated ancillary buildings or by backstaying to foundations designed for the purpose.

A sagging rope is outstandingly capable of carrying permanent loads because equilibrium is reached with the catenary curve, but the addition of unbalanced working loads (Fig. 26) brings about a new state of equilibrium which is accompanied by marked changes in shape. Even wind loading of the roof surface, which is predominantly suction, causes a change in the state of equilibrium and, where the dead load is slight, may lead to a slackening of the supporting ropes. Lastly, vibration and flutter may occur and cause damage to the roof sheeting. These unwelcome consequences can be excluded if tensioned ropes are laid over the array of main supporting ropes and at right angles to these, so that a roof of saddle form results (Fig. 27). The

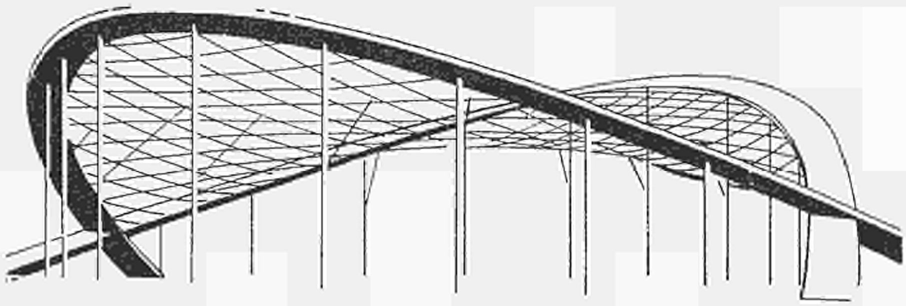


Fig. 25

tensioned ropes, by virtue of the vertical components they produce, not only increase the aownward loading at each intersection of the tensioned ropes, but, by the reversed suspensiion arrangement, conttribute greatly ot the stabflity of the system under wind suction and unbalanced loading. The statics of the bridge in tubular construction (13), depends on the same principle.

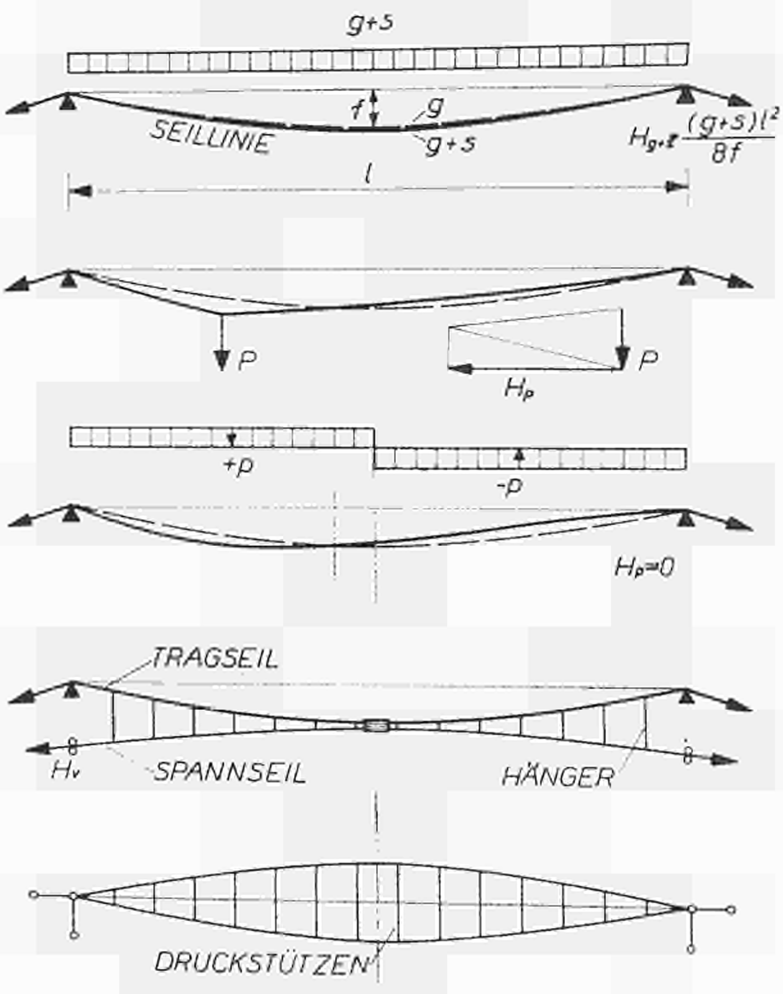


Fig. 26

The amount of steel used in suspension systems like these is, of course, relatively small, but it is nevertheless very high strenght steel. In many cases it is only by adopting such methods that competition from shell roofs in reinforced concrete can be met.

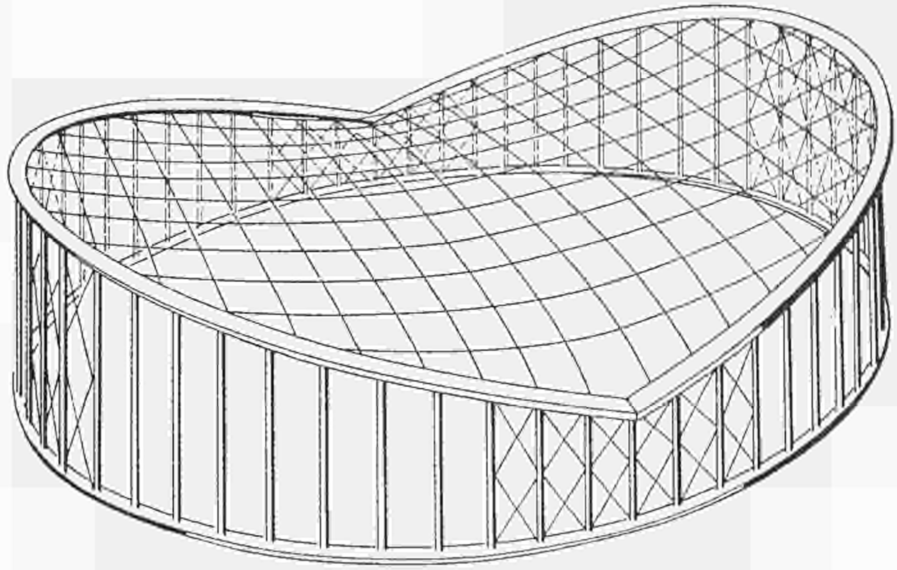


Fig. 27

Weir Gates

At the Structural Steelwork Conference of 1962, I reported briefly on the use of shell constructions for weir gates in hydraulic engineering work. In Austria two basically different types of sluice board are used, namely the troughed type of sheeting and the shell type. With the troughed facing (Fig. 28), the wall sheeting, which forms in itself the flange of the main girder on the impounded water side, has its units cambered slightly in a horizontal direction into the water, so that a half trough occurs in each bay (Fig. 29). Since the outward facing flanges of the main girder are shaped to a catenary curve, the compressive forces in the upstream flange plates developed by the girder under load are uniform. In the slightly curved plates, certain secondary forces are thereby set up and are resisted by the water pressure, so that the diaphragm action of the weir plates is only called on to take minor resultant forces. Therefore both the plate thickness and the number of stiffeners can be quite small.

The shell-type dam wall (Fig. 30) makes use of the strength characteristics of the self-spanning barrel shell for carrying loading that is distributed over its surface. As has been proved by thorough theoretical investigation, this kind of shell really absorbs the water pressure by membrane stressing. The irregular edge deformations (Fig. 31), which arise from non-compliance with the general conditions of strain at the edges of the shell (longitudinal and transverse beams), are really too slight to enter in the calculations.

By the judicious use of cambered dam facings it is possible to reduce the weight of weir sluices by about 15%, and the omission of a heavy system of stiffeners makes for economy in shop fabrication.

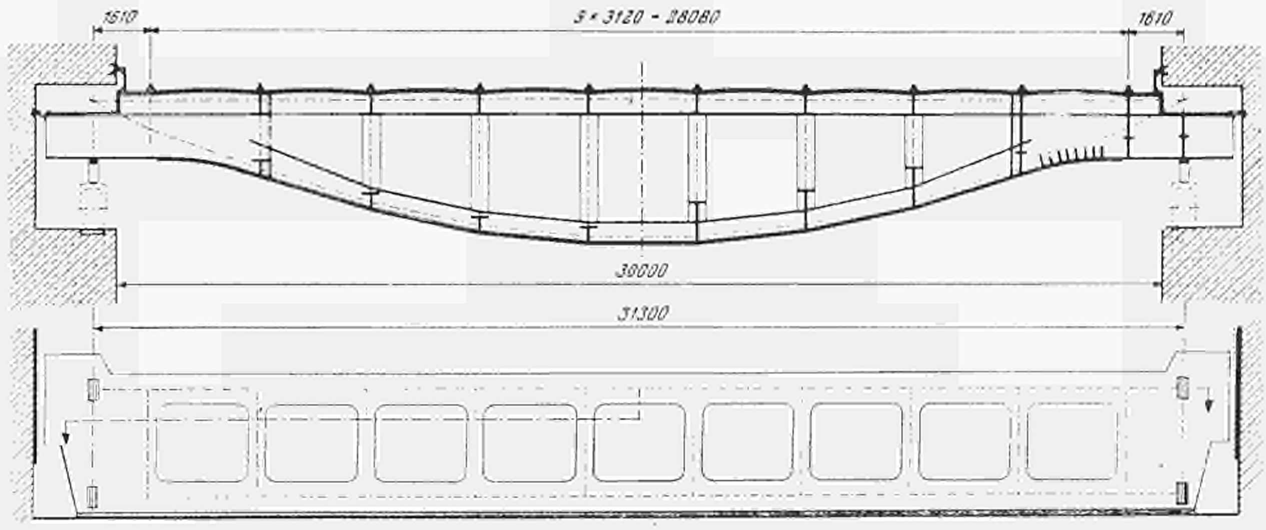

1610
9 × 3120 = 28080
1610
38000
31300

Fig. 28

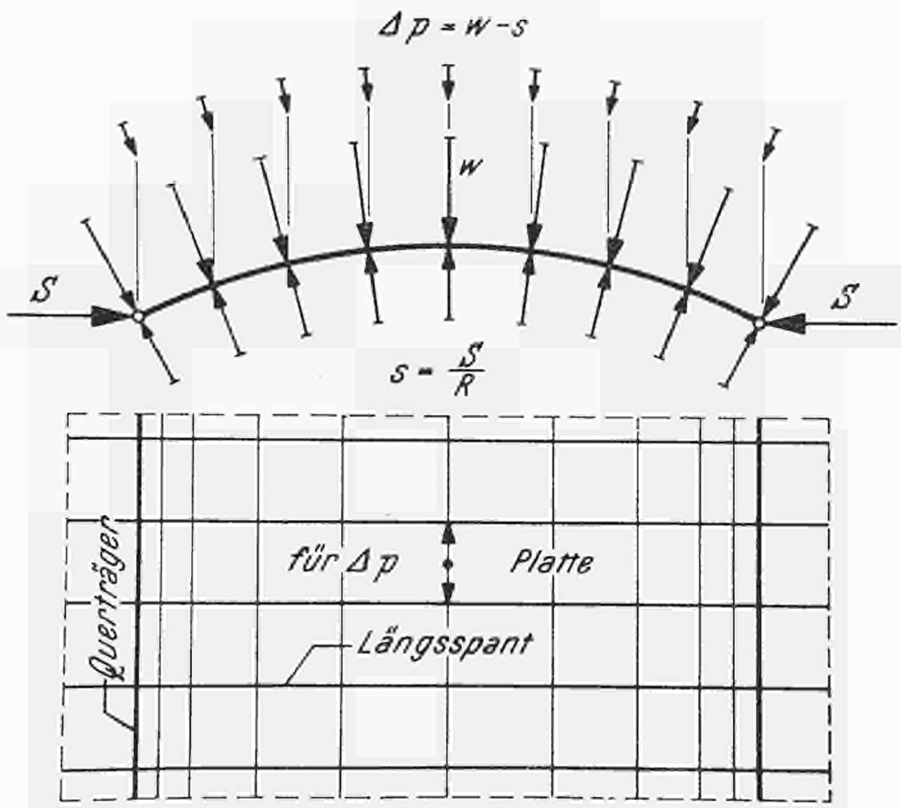

Δp = w - s
w
S
S
s = S/R
Querträger
für Δp
Platte
Längsspant

Fig. 29

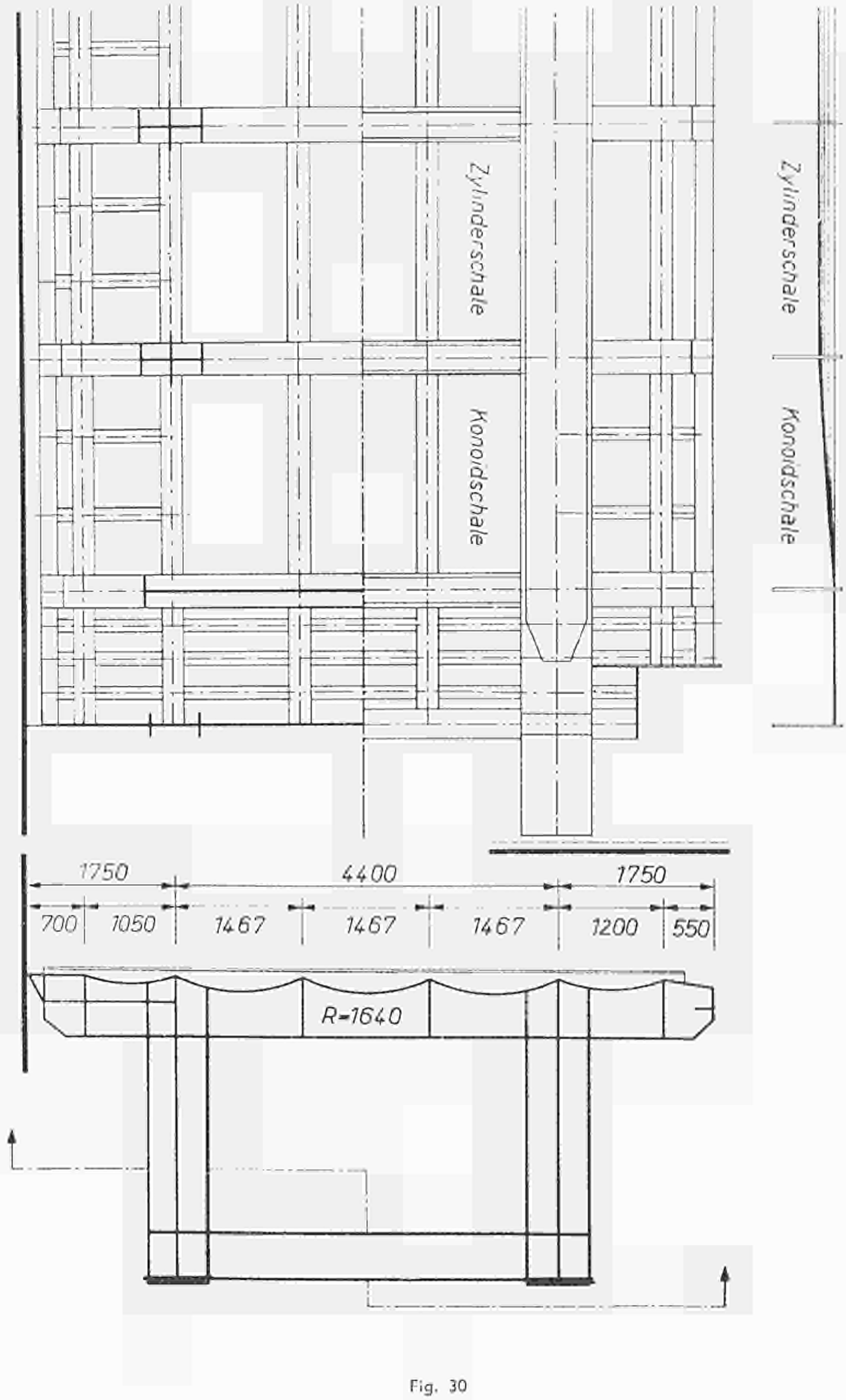

Zylinderschale
Konoidschale
Zylinderschale
Konoidschale
1750
4400
1750
700
1050
1467
1467
1467
1200
550
R=1640

Fig. 30

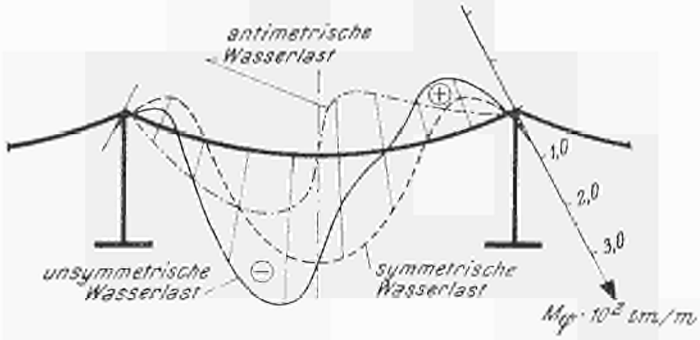


Fig. 31

Problems of buckling and codes of practice

Any measures taken to ensure adequate stability in structures play a decisive part in dimensioning, practical construction and in the amount of steel used. The basis for any understanding of the stability of a structure is given by the curve that shows permissible buckling stresses for various slenderness ratios of a straight bar with hinged ends under axial thrust. Figure 32 reproduces the curves of permissible buckling stresses taken from the codes of practice of different countries. The first thing one notices is the great differences between them. For example, the French buckling stress curve in the principal range of slenderness ratios gives the highest permissible values. The cause of such wide variations lies in the uncertainty of our knowledge on inevitable departures from the perfect theoretical assumptions on which the plotting of these curves of buckling stress depends. Such inaccuracies are due to variations in modulus of elasticity and plastic yield point, the residual stresses left by welding and rolling, curvature of the axis of the bar and eccentric application of forces. These are recognized as structural and geometric imperfections.

The present state of standard practice as it affects the theory of stability is notable because of the collaboration throughout Europe of the various structural steelwork institutes, which has resulted in the formation of "Committee 8". This Committee is occupied in working out European codes for obtaining a complete body of information on the stability of steel structures. These problems cannot, however, be solved by theoretical examination alone. Tests must be carried out in the laboratory. This is because even when the effects of all imperfections are known, it is still extremely difficult to assess the load-carrying strength of a member by theoretical methods alone. Also these imperfections are not in the nature of fixed quantities, but are subject to a variation or scatter which can only be dealt with by statistical methods. A two-fold task awaits laboratory experiment, namely, the checking of the load capacity as determined by theory and the preparation of statistical information regarding imperfections, which must be supplemented by observations on executed structures.

European co-operation in this work is commendable. Up to date more than 300 tests on the bars themselves, of various sections and various slenderness ratios as commonly incorporated in structures, have been performed in Belgium, Germany, France, Jugoslavia and latterly in England and Italy. The evaluation of the results so obtained by theories of probability will in conjunction with a collapse-load theory for struts with normal imperfections give us a new curve of buckling stresses which will represent as realistically as possible the performance of struts in any structure (14).

The serious effects of residual stresses (Figs. 33 and 34) on buckling strength and on the dependence of buckling loads on sectional form make it advisable to establish, not only a set of values for all kinds of sections, but two or three sets of values.

The question of security of structures against failure demands very exhaustive study. Formerly it was customary to apply to the buckling stress values a fixed co-efficient or one that varied with the slenderness of the bar, and then to use the permissible values so obtained for fixing the dimensions of the required axially

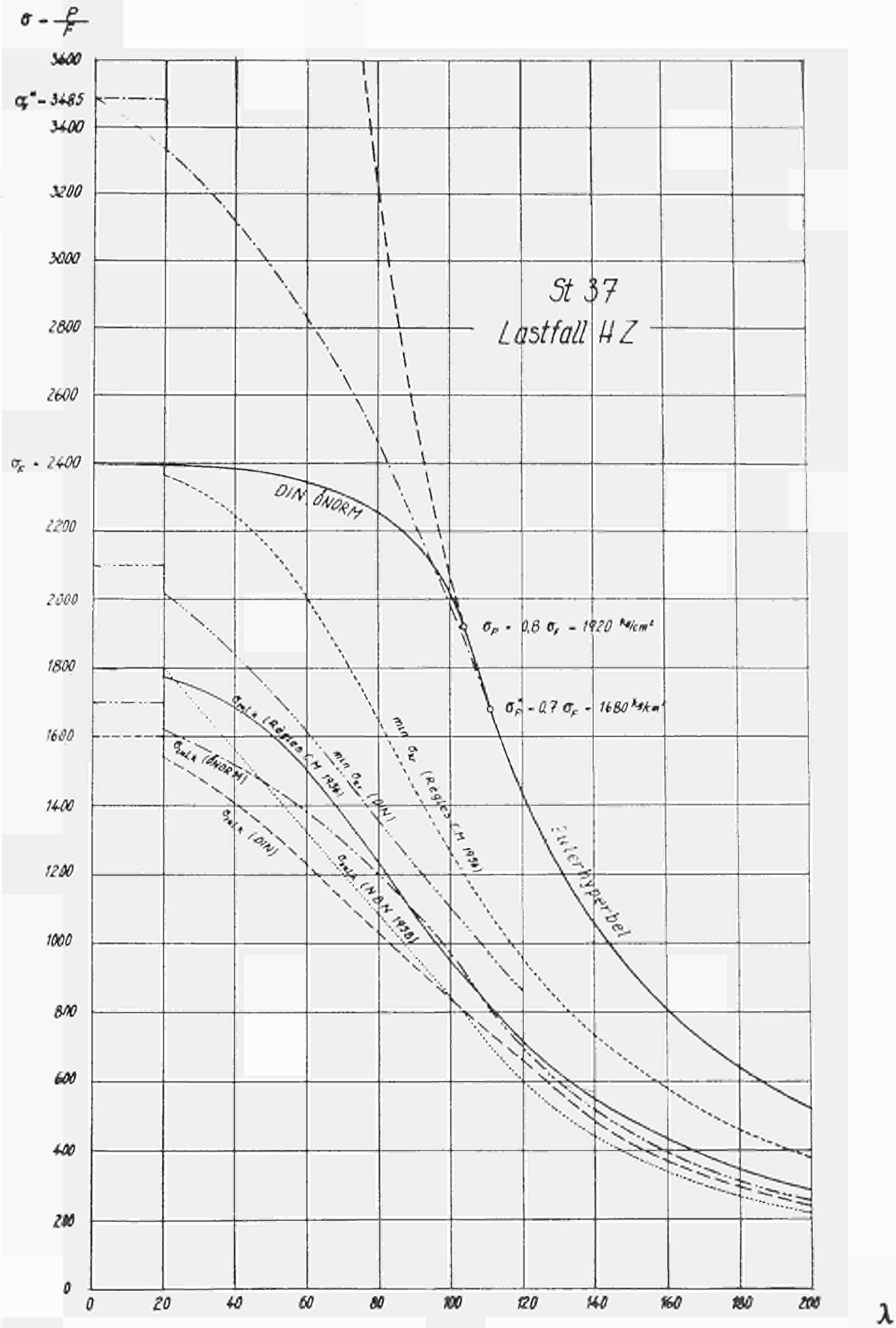

σ = P/F
3600
3400
3200
3000
2800
2600
2200
2000
1800
1600
1400
1200
1000
800
600
400
200
0
20
40
60
80
100
120
140
160
180
200
λ
St 37
Lastfall HZ
DIN ÖNORM
Eulerhyperbel

Fig. 32

loaded strut. If one works from the strut, with its inherent imperfections, and accepts the lower limiting curve as the appropriate buckling-stress curve, below which in practice there would be little likelihood of failure, (Fig. 35), then one has only to apply a coefficient of safety to take care of the uncertainty of loading.

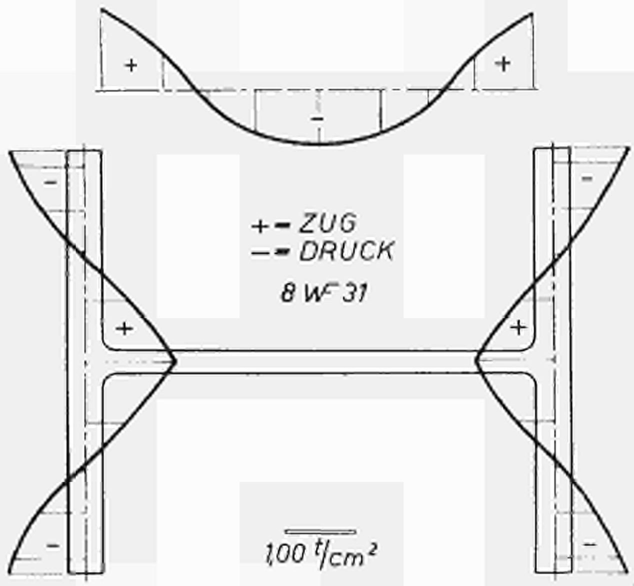


Fig. 33

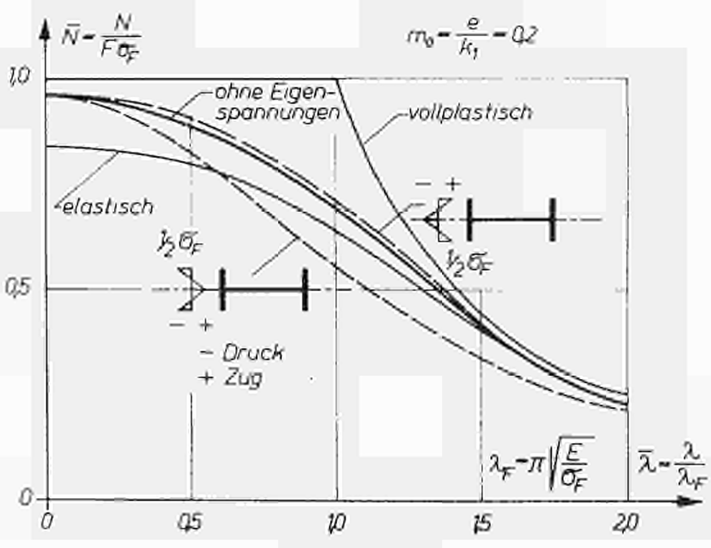


Fig. 34

Thus it is necessary to give factors of uncertainty to the different kinds of loading, such as permanent loads, superimposed loads in use, snow loading and wind pressure and any other stress-inducing factors. These factors will vary with current regulations which prescribe loading and with the nature of the loading and will generally, but not always, be greater than unity. For this reason one needs to have access to adequate statistical data in order to arrive at a coherent understanding of these factors of uncertainty after evaluating those data by theories of probability.

The question of a margin of safety for complete structures is thereby solved, since the case of the individual member can be extended to cover the problem of overall stability. Similar methods suggest themselves for bars in tension, for here the permissible stress depends on the yield point that is likely to be maintained with the same degree of probability as the collapse-load stress in the case of buckling. Bending and torsional stressing, and also multi-axial states of stress, in plate and shell type structures each call for special consideration of the limiting stresses. In dealing with the structure as a whole one would naturally take into account the strength that remains in the plastic range, both for the system and in its component sections, in order to arrive at a factor of safety equal to that chosen for the bars in tension or compression. It is not possible here to deal with the analysis of structures by the plastic theory.

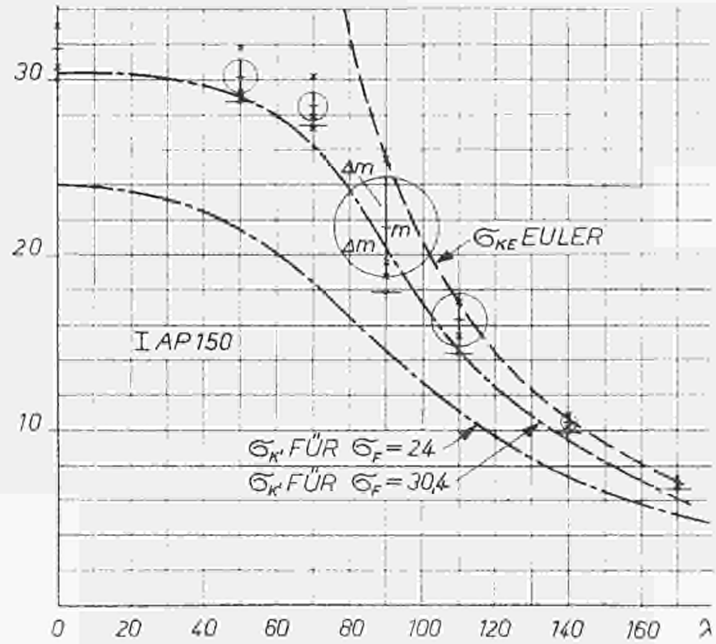
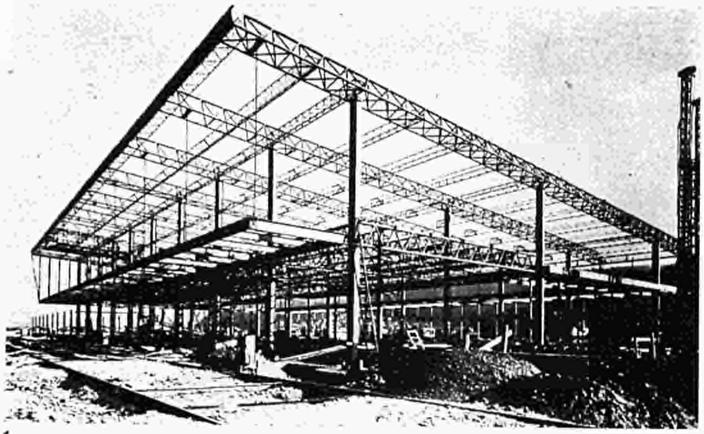


Fig. 35

Concluding Remarks

If it is to survive the fierce competitive war with other forms of building, steel construction must for ever be blazing new trails and trying out new solutions. Here, theory, fabrication and erection should work hand in hand to attain the utmost in security, efficiency, economy and beauty. Modern ideas favour thin-walled construction in lattice or stressed-skin structures, space frames, plate and shell type structures and structures wherein ropes in high-grade steel act as ties and tubular sections act as struts.



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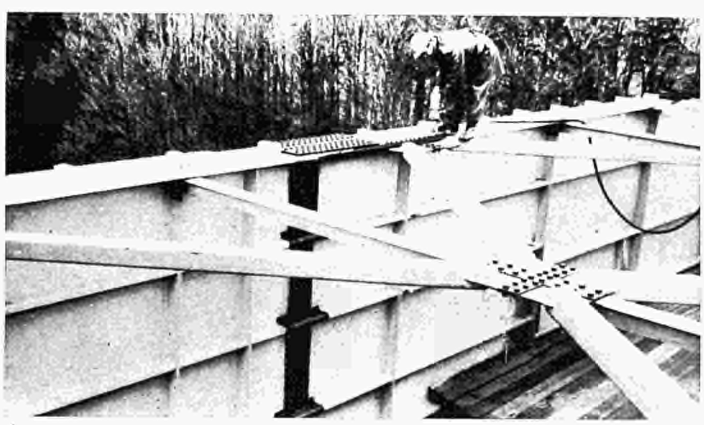

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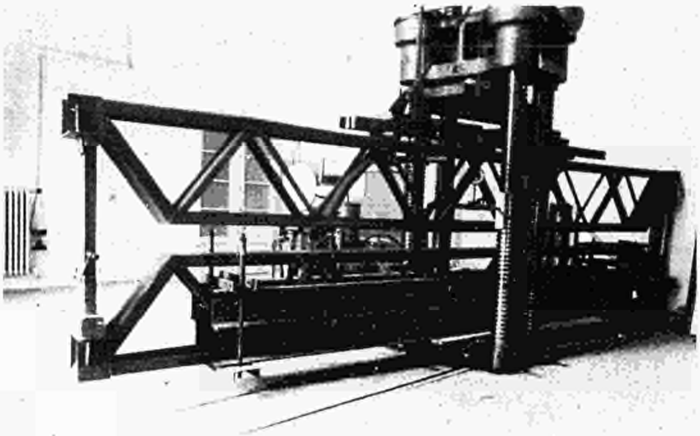
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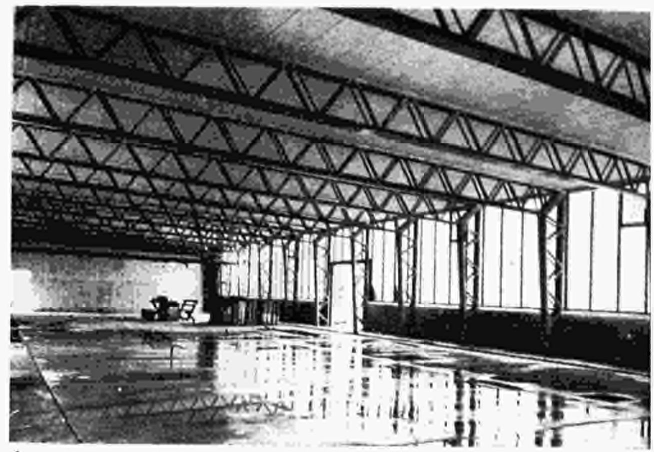
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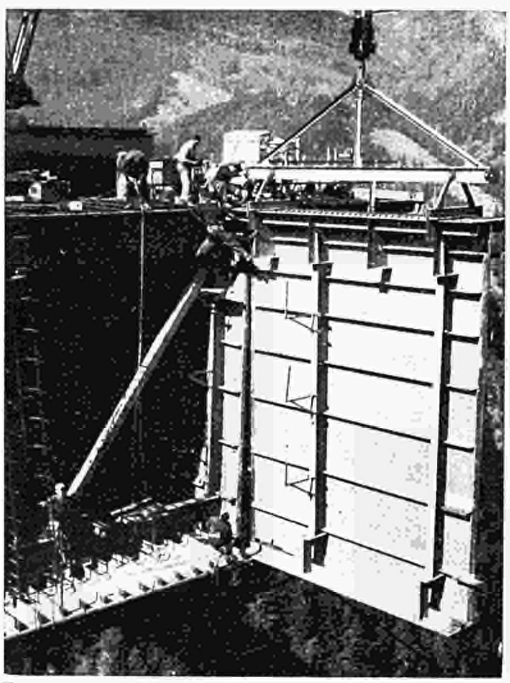
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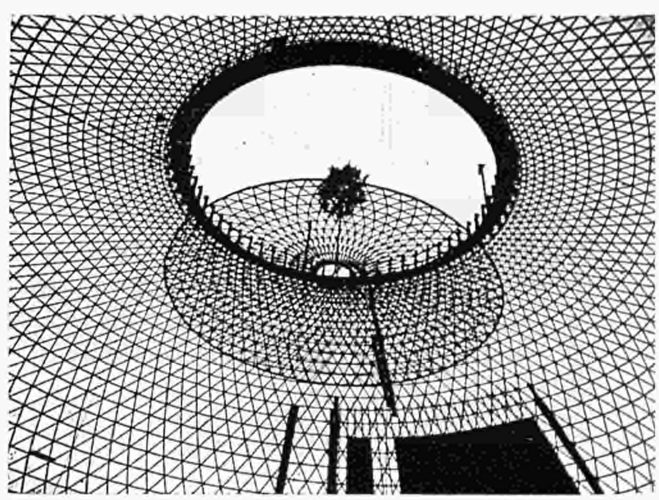
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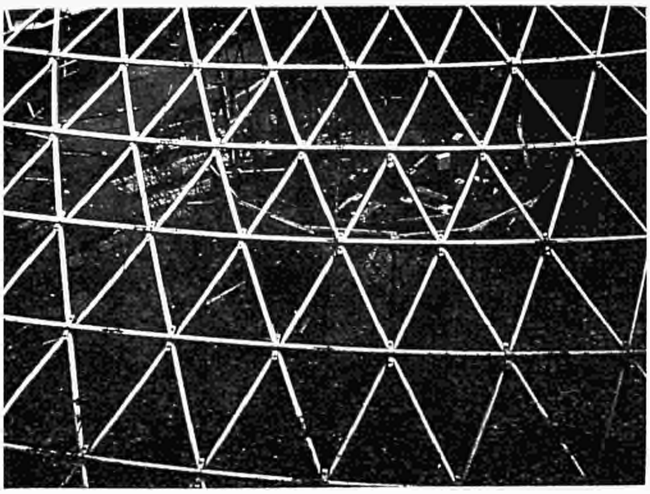

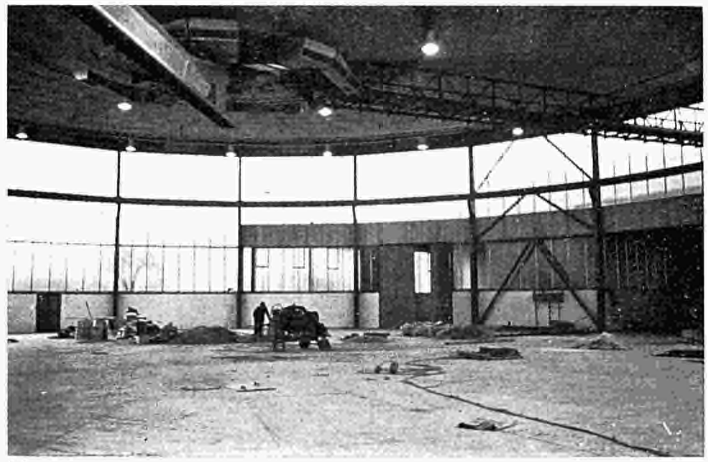

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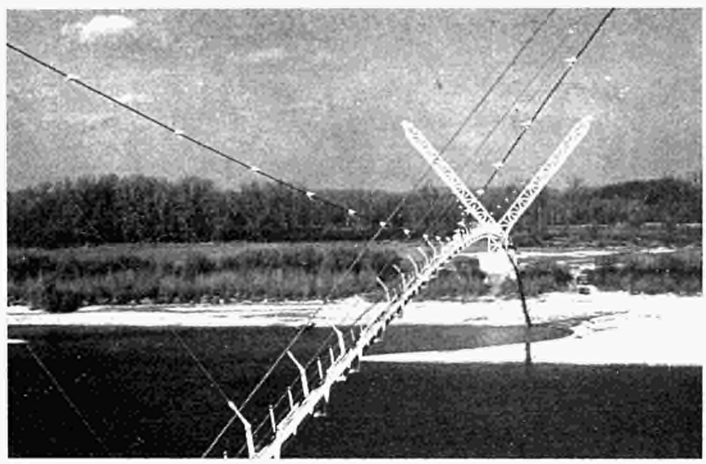

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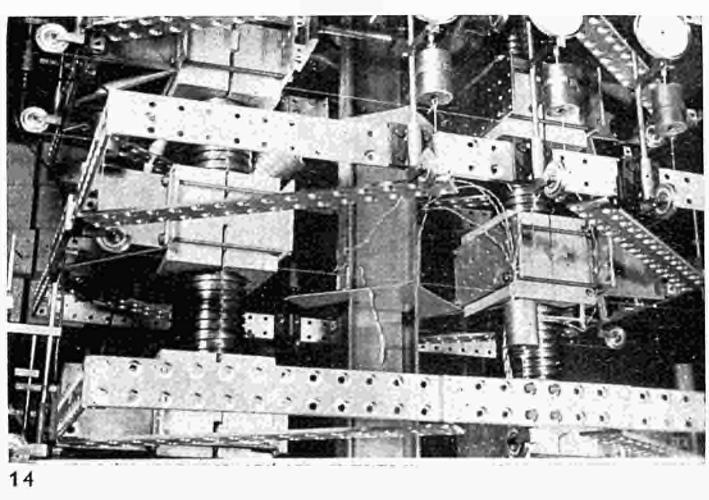

14

Henri LOUIS

Design and Calculation of Steel Construction

(Original text: French)

As the time available for this general introductory talk is rather short, it seems best to use it for bringing forward or recalling a few ideas which, if all of them could be made more or less simultaneously present in the minds of planners, and if they were to be applied with due understanding, might serve to raise the design of steel structures out of the rut or to free it from the bonds of outworn tradition.

Drawing offices engaged in designing steel bridges and other structures do not, as far as their engineers are concerned, make sufficient use if any at all, of the means and methods of calculation which have already been made available to them. Yet in some countries experience has shown that the mere utilizing of these means and methods makes it possible to affirm the suitability of steel and to adopt it instead of other materials, with success, or to promote and generalize its use in these types of construction.

Bridge decks provide a striking example of this, for the theory of slabs presenting resistance orthogonally from all sides (*dalles orthotropes*) is now within the reach of everyone, thanks, in particular, to the remarkable work of our Chairman, Professor Pelikan, and although this clearly is not a solution to be adopted indiscriminately such a slab ought not to be ruled out, as it sometimes is, merely because its calculation differs from that of a simple supported beam, but only because it is believed too expensive to build. However, in my opinion, that belief is rarely justified; for on the one hand the design is not stereotyped and the designer may find scope for his ingenuity in seeking a solution that is at once rational and economic, and on the other hand the „orthotropic slab" has a very considerable carrying capacity due to two features : its action is similar to that of a diaphragm, and it plays at least two simulatenous rôles in the behaviour of the structure. Consequently, both in itself and in its contribution to the whole, it is a means towards economy in the design of a homogeneous steel structure.

This raises the point that while in fact many components of the structures we build fulfil several functions simultaneously, this is all too rarely taken into account when contriving and designing these structures.

In a large bridge where the main girders serve also as parapets, the longitudinal beams under the carriageway can easily be arranged so as to support localized loads by the provision of bending resistance between the intermediate supports, at the same time as helping to resist stresses in the structure as a whole by actively co-operating with the tension chords of the main girders and also serving as lower bracing.

If due account is taken of the functions which these longitudinal members actually fulfil—even though the designer may arbitrarily have decided otherwise—instead of making the traditional assumption that they serve no other purpose than supporting local loads, this does not mean the longitudinals need be made any heavier: on the contrary, it may be possible to lighten them by choosing a higher value for their modulus of resistance to bending, while at the same time the cross section of the tension chords (of the main girders) can be made notably smaller. Thus, bottom-bracing can be entirely eliminated, the connections become simpler and therefore less expensive, and finally—what is always desirable—the appearance is improved.

Civil engineering structures have three dimensions : it is more than ever necessary to recognize this and to realize that traditional methods of calculation seriously underestimate the load-resisting capacity offered by at least the greater part of the constituent elements.

I shall cite only one piece of evidence for this: in a steel bridge across the Meuse at Liège, subjected to rigorous tests in which the design loads were realized, the stresses in the supporting members under the effect of those loads were systematically measured and amounted to only 2-3 kg./mm². (1.3-1.9 tons/in².).

While theories of space structures are not yet worked out or well enough established for all cases, the results obtained from model experiments can be of very great help. They make it possible to establish methods of calculation which although only approximate are nevertheless fully valid.

The models can be devised in accordance with the kind of information it is desired to obtain; as regards the behaviour of structures under working loads and as regards the mode of failure they can be made to yield information unobtainable by analytical methods of calculation. The technique of experimenting with models is far too little used in the field of engineering construction, and through failure to use it, the opportunity of trying out the merits of new ideas and new forms is lost.

Structures are all too often designed on the basis of some solution chosen *a priori*, despite the fact that other solutions are possible; solutions at least deserving of critical and numerical examination before a choice is made. Moreover, dimensions arrived at on the basis of the one adopted solution are not always the most rational; still less frequently will they prove to be the most economical. Too many simplified assumptions are made merely for the sake of ease of calculation when the use of electronic computers would enable the problems to be solved on a more realistic basis. Moreover, and above all, it is possible, using these modern methods of calculation, either to arrive at the optimum dimensions in relation to the chosen solution or to take cognizance of the qualities offered by any one of the other contemplated solutions. The cost of using electronic computers is not high in relation to the time they save in the investigation of projects, the technical advantages gained, the assurance of more accurate calculation and, above all, the economy of the resulting construction.

The pursuit of safety is becoming ever more important and will necessarily affect the planning and design of structures.

The irrationality of designing by reference to what are called "permissible stresses" is becoming ever more manifest although this method is still widely used. The predetermined values of these stresses are closely related to the live load values, likewise stipulated by the regulations.

Without pursuing any further a problem which is already being widely discussed, let us note that while a trend to consider safety from the point of view of probability is becoming clearly apparent, the principle underlying it has scarcely been examined, so far as steel construction is concerned, except as regards the material itself. The application of the principle is not being extended, as it obviously might be, at least to building components if not to structures as a whole. It ought in any case to be considered as a means of agreeing on realistic values for the live loads on which there are very few data to go upon. It is, indeed, paradoxical to design a structure for overloads it will almost never have to carry even under test, and particularly to make no systematic distinction between structures of small, medium and long span. These loads ought to be defined in terms of their probability, a procedure which would assuredly lead to substantial savings as well as being in accordance with good sense and with reality.

Plasticity calculation has been widely developed, at least in theory and on the basis of laboratory experiments, but in Europe it is scarcely used at all, even for structures as familiar as roofing of the traditional type. Yet some American rules dating from the last two or three years seem to place a degree of confidence in plastic design equal to that which most engineers still place in designs based on the elastic theory.

Distrust of plastic theory is not founded solely on arguments of a psychological kind. At the present time it is true, indeed, that this mode of design does have its weak points, especially as regards uncertainty as to residual stresses and as to the true value of the yield point of the material, or again as to the hypothesis, often adopted in this method of design, that no consideration need be given to the phenomenon of brittle fracture.

Nevertheless, it would be wrong not to recognize that on the one hand designing in terms of plasticity comes closer to the real "working" conditions of a structural component or those of a structure as a whole, and that on the other hand the limits of stress to which it points are almost automatically reduced by the absolute necessity of not exceeding acceptable amounts of deformations, which can be predetermined.

Nor are the defects inherent in the usual elastic method of design any smaller. At any cost, a conflict between elastic and plastic calculation must be avoided. I sincerely believe that the two can be made complementary to each other, each being a useful and valuable tool.

The fundamental problems always present, are those of safety and economy. In this context it would seem that for many types of construction the method of design and the corresponding degree of safety—even if this can only be thought of subjectively—might be varied from one element of the structure to another, due regard being paid to the nature of that element, the stresses in it and the consequences, which its failure would entail upon the stability of the work as a whole, while always safeguarding life and property.

As regards modern types of bridges, it should be remembered that, in direct contrast with the attitude prevalent not long ago when roads were expressly so located that bridges for them could be built of entirely conventional type and nearly always straight, nowadays it is the bridge that has to be adapted to the layout of modern fast-traffic roads.

Consequently, in several countries many bridges erected or in course of erection are skew, curved or even on a spiral alignment; but in other countries such structures are rare or even non-existent.

These bridges are nearly always of the deck type, designed as frames made up of beams or box girders.

Methods of calculation and collections of influence lines have been established for skew bridges and entire books have been devoted to them, so that as a rule they can be designed in the average drawing office, particularly if the bridges are grids of beams. For skew box-bridges, some very interesting investigations and experiments presented at the last congress of the International Association for Bridge and Structural Engineering point to the advantages of skewed ends which, amounting as they do to the provision of end restraint, have the effect of reducing the mid-span moments, always provided that the ends are sufficiently stiffened and braced to transmit the forces into the box section.

With curved bridges, too, it can be said that these are easily applicable theories based upon the general theories of stability, particularly the equations of Bresse, whereby influence lines and influence surfaces for torsional moments can be plotted; the bending moments are almost the same and the shear forces exactly the same as in a simply supported beam.

I believe that curved box-girder bridges of steel are especially advantageous because the slab above, already stressed in a special way by reason of the double rôle it plays, can withstand considerable shear forces.

Box-girder bridges are far too rarely adopted, even on straight alignments, because some designers—wrongly in our opinion—consider them to be too expensive: for this reason it is difficult to understand why in some countries so many bridges of this type are built.

Technically and economically this is an attractive solution when it is remembered, with good reason, how advantageous is the "orthotropic slab."

Such a bridge may be built with one, two or a multiplicity of cells. If the girders are multiple the question of interconnecting them arises : the best and most economical solution would seem to lie in providing a number of rigid braces at the same level as the box girders. It is possible, although it is a complicated task, to design these braces by calculation ; the Belgian commission for structural steelwork has published a design theory for them which has been successfully applied and which has been confirmed by a model experiment.

Another problem in designing box-girder bridges is that of deciding what effective width of flange to assume. In some road bridge designs the moment of inertia of the cross section has been calculated, without particular difficulty as including the total width of the flange plates above the two sides of the box and connecting them together. German and Austrian engineers who recently carried out tests on box-girder railway bridges however have shown that a smaller effective width can be assumed for flanges, especially in the case of concentrated loads. This means that the flanges immediately adjoining the webs of the box girders should be strengthened.

To conclude these considerations, it can be stated that skew bridges, curved bridges and bridges with one or more box girders have become accepted solutions. There is no need to avoid them and thereby impair the performance of a traffic route alignment, still less to make the calculations for them depend on expedients and hypotheses which, when all is said and done, result in less safety at greater expense.

Now I shall proceed to break a lance in support of the lattice girder bridge, which is an especially attractive solution from the economic standpoint but which in some countries is systematically avoided by reason of its appearance.

This objection can be countered by choosing the right type of lattice for the particular case. I am thinking here particularly of the type that has diagonals but no upright member, which has frequently and very successfully been erected in Germany and France. It should be combined with a better choice of cross section for the tension bars, with less prominent nodal connections (these are easy enough to design) and with a more rational overall design of the work of which the lattice girder forms part, thus attaining greater economy of construction through the use of one or more prestressing operations which make for lightness and improved appearance.

I had occasions quite recently, to deal with the problem of building a deck bridge (one with the deck carried above the girders) of 60 m. span and 16 m. width to carry a motorway. The preliminary design—entrusted to a young engineer who had graduated a few weeks before, as the final exercise in his course of study—was carried out under my direction and under the supervision of a chief engineer at the Centre Belge de Recherches de l'Industrie des Fabrications Metalliques (CRIF).

It is impossible here to go into details of this work but it may be stated that the structure comprises six identical lattice girders 3.70 m. high, inclined away from the vertical towards one another; the upper slab is of concrete and contributes to the strenght of the whole; provision is made for adjusting the forces acting in the bridge at the time it is built; prestressing has been carried out in two stages; all the lattice members are tubular or semi-tubular.

It must be remembered that this is a preliminary design, or rather the trying out of a new principle, so that a more detailed calculation, bearing in mind that the structure in question is a lattice box girder, would have led to a saving. This being so, the steel required for the work weighs 107 kg. per square metre of deck area, that area being calculated for the whole width of 16 m. which includes two cantilevered footways 2 m. wide. The structure is of "A 52 a" grade steel, and as already stated it is 3.70 m. high; the width of its lower part is 8 m., and it is entirely welded.

Further details of this example could be given in the course of the discussion. It is imperfect both in calculation and in planning, but it does exhibit a tendency which is technically feasible. The saving finally achieved remains to be worked out, and the question of appearance deserves to be examined in co-operation between an engineer and an architect both animated by the urge to achieve the best all-round solution.

This case illustrates the three-dimensional principle we have contemplated, but a further justification for "space lattices" lies in using them to cover large areas. Their use has been proposed by certain forward-looking architects and engineers, but such structures have been far too little developed, perhaps because of the uncertainties involved in calculating them, but mainly I think of the unwarranted misgivings entertained on the subject.

As Dr. Ingenieur Makowski says in his book "Constructions spatiales en acier"—which every engineer and architect ought to read—"using steel in their construction adds further to the advantages of space structures, which can be considered as an extension of traditional lattice (or truss) systems into three dimensions." The advantage offered by these structures is that lines of action of the forces are ramified in three dimensions with the consequence that a homogeneous field for the play of those forces is produced. This increases resistance and allows the cross sections to be reduced, with considerable saving of material, the lightness and shape of the structures giving them a most attractive appearance.

In the planning of space structures, imagination and boldness can almost be unrestrained, guided as they can and ought to be by auscultation in model testing, which is easy to perform.

Steel can claim to be far superior in this field; whether for the purpose of barrel vaults, cupolas, folded plate roof construction or suspended roofs. This superiority is all the more marked by reason of the fact that space structures made of steel lend themselves particularly well to the use of plastic material as a covering, the combination of this with steel being certainly a field of the future.

Passing to another subject, I should like to say a few words about composite construction in steel and concrete, a way of building already well known but too seldom adopted by engineers although in many cases technically satisfactory and economical because both materials are used in a rational way : the concrete serves two purposes, the execution is easy and it is possible for the roadway slab to be prefabricated.

Occasionally some prejudice against this is encountered because steel constructions must necessarily be homogeneous in view of their behaviour. I do not think this reason is a valid one, even though the action of the two materials in relation to each other is not yet perfectly understood as the phenomenon of differential deformation by the concrete comes into play and for this reason the value of the modular ratio is subject to some inaccuracy. Moreover, promoting the use of steel construction ought not to mean using this material indiscriminately; for the modern user this high-grade and correspondingly expensive material is one to be adopted in situations where it is necessary and where it provides a technically and economically sound answer. I am convinced that if, in some countries, steel construction has actually declined it is because steel has been used without sufficient discernment.

A first application of composite construction is the adoption in bridge construction of precast slabs for the deck, having underneath them an arched steel sheet onto which a 5-10 cm. thick layer of concrete is poured, shock resistance and bonding being ensured by a steel reinforcement fabric or by plates welded onto the sheet. Many such applications have successfully taken place in France and Belgium. For use in the construction of floors in buildings the composite slab is made by starting with a thin steel sheet bent in various ways to serve both as shuttering and reinforcement; the saving is in the cost of the floor reinforcement that would be needed for reinforced concrete construction of the ordinary kind.

Bridges of composite construction are now quite common : in these the deck is above the girders and consists of a reinforced or prestressed concrete slab which contributes to the strength of the bridge as a whole, either naturally in the case of bridges spanning simply between two supports or by one of those devices for adjusting the play of forces that are made possible in several ways by the use of partial or total prestressing.

Such an arrangement leads to a considerable saving of steel in beam-grid bridges or to a saving which may be less evident, though probably real enough, in the case of box girders where the merits of an „orthotropic" steel slab need to be balanced against those of a reinforced or prestressed concrete slab.

Thorough investigations and research have been carried out on the subject of composite structures, and these can be put to direct use although their application involves some difficulties. At the same time approximate methods of calculation have been proposed for practical use; these give a satisfactory degree of accuracy, and the dimensions worked out from them show a substantial saving.

The method of bonding the concrete to the steel and the quality of the bond are two important questions to which many solutions have been proposed, particularly with a view to prefabrication of the slabs. Professor Dr. Ing. K. Sattler of the College of advanced technology in Graz, who is a leading specialist on composite structures, has recently suggested and designed composite precast concrete girders in which the bond is achieved by means of studs welded to the steel and able to be anchored into the concrete slabs. Openings are provided in the latter to receive the studs and later filled with a special mortar, while the various elements within the slab are bonded together according to thickness.

In certain German and Belgian structures the slab, cast in situ is bonded to the webs of the steel beams by transverse prestressing while shear forces are taken up by friction.

In the building industry some half-hearted applications comprising the use of thin tubular steel columns, circular or polygonal, filled with concrete have been made. No doubt research is still needed on this matter, especially with a view to determining the modular ratio and studying the effect of friction on the enclosing sheet metal. This procedure is certainly a promising one; it must lead to a saving in space due to the reduced cross section of the columns, along with lightness, great safety and a saving in cost; moreover it would seem at first sight to allow of esay connections between columns and beams.

As regards traffic possibilities, the future of our towns is bound up with the construction of elevated roads whose contact with the ground must be through almost point-loaded supports. Up to now, steel construction has been peculiarly absent from works of this kind although the box principle, or this in the form of hollow steel fabrications, offers an excellent solution. It is certain that for reasons of appearance and lightness there will be an increased use of mushroom-slab bridges supported on a single line of columns; but in this respect nothing has been built up to now of steel—though a steel sheet conjoined with a small thickness of concrete, or perhaps a thin concrete slab sandwiched between two steel sheets, suggest themselves as answers deserving at least of examination.

The use of steels with a very high yield point is in my opinion, also a very important factor in the progress of bridge and structural engineering. The use of such steels will assuredly be one of the most helpful means of competition with other constructional materials, though in our western European countries, if we are correctly informed, their use has been contemplated only for pressure pipes and tanks. Such steels should not, of course, be used for all structural components; they would appear to be most useful for elements subjected to tensile stress.

Further, as these steels have the same modulus of elasticity as ordinary steels, the need to limit the amount of deformation may hinder their adoption, but it is this very problem that might apparently be solved by using steel jointly with ordinary or prestressed concrete.

The employment of these steels would raise certain other problems, notably that of their welding, but such questions can be answered by scientific or practical means since the difficulty is usually psychological and is caused by some obsessive fear of brittle fracture. These problems can be disposed of only by full co-operation between metallurgists, producers of welding rods and equipment, and designers.

It is regrettable that although we now have very good steels at our disposal with yield points ranging from 40 to 80 kg./mm^2. (about 25 to 50 tons per square inch) these are scarcely used at all in structural engineering, while at the same time steels with a breaking stress of more than 150 kg./mm^2. (95 tons per square inch) are being used in prestressed concrete under working stresses in excess of 100 kg./mm^2. (63 tons per square inch), the safety of the construction being dependent entirely on the behaviour of the steel.

Prestressing has already been much employed in conjunction with steel construction, chiefly in bridges and especially for continuous girders. Close to the supports, the cables are attached to the upper chord of the steel superstructure after the concreting of the slab which, of course, is left unfilled immediately around the cables, these being covered after tensioning so as to complete the monolithic action of the concrete.

Research carried out, more particularly in the U.S.S.R., shows that designing prestressed welded or rolled beams in terms of plasticity saves some 10 to 15% of steel as against designing in terms of elasticity. In this context it seems to me that prestressing should constitute a further justification for plastic design, always provided that due account is taken of fatigue stresses and of the brittle fracture problem.

It is noteworthy that prestressing can be looked upon in a wider sense as a means of adjusting the forces engaged, so as to bring about a redistribution of the stresses due to dead load and permanently applied loading, at the same time meeting certain necessities such as in regard to the available headroom, or so as to help in improving the appearance of the structures. It should also be noted that adjusting the play of forces does not necessarily result in a saving.

In regard to the shape of the sections used in steel construction it may be permissible to voice some dissatisfaction : it is rare to find shapes suitable for welding and generally it is the sections intended for riveted work that are used. We can only pray that as soon as possible something practical may emerge from certain researches that have been undertaken with a view to producing steel sections formed by extrusion. This could amount to something quite revolutionary.

The designer along with the steelmaker ought to be able to focus his attention on developing sections made of bent sheet. These exist, or are possible, but there is not yet any confident knowledge of how to use them. It is essential that drawing offices should go all out to find rational ways of using these sections in a great variety of shapes, for until there has been a complete rethinking of structural skeletons or frames in terms of these shapes—particularly as regards the connections and nodal points—the considerable potential value attaching to sections of bent sheet, used either by themselves or jointly with concrete, will not be exploited. The large-scale use of tubular sections fabricated from bent sheet, after thorough investigation, would open up great possibilities. Structural steel assemblies, especially if welded, have not received enough attention as regards either their form or their calculation; hence many assemblies are over-dimensioned, which may create a false sense of safety or even be dangerous.

While this is true as regards the calculation of static stresses the opposite may apply if the assembly is subject to fatigue stresses, for the true permissible stress in the assembled pieces may in fact be greatly reduced if the welded joints are inefficient.

A passing reference may be made to the considerable interest attaching to connections made with high-strength bolts. Much research has been or is being carried out and points to great possibilities, although there are countries in which such bolts have been very little used, if at all.

Welding is not being used to take sufficient advantage of very thick sections, either because their mechanical properties diminish in value at a rapid rate or because it is feared they are liable to brittle fracture; often, however, this fear is unfounded if high-grade steels are used and if the welding is correctly done.

Convinced as I am, of the advantages and potentialities of welded construction, I do not feel I am doing it any disservice by asserting that for large structures and for those intended to serve very important purposes a wholly monolithic welded construction is not necessarily the best.

I am of the opinion that in structures of that description many difficulties might be circumvented by adopting welded joints as well as joints secured by high-strength bolts. This applies particularly when dealing with the phenomenon of brittleness, which we may remember as constituting a serious obstacle to the design of welded frames by the plastic theory method. It should morever, be noted that this solution is often the one chosen in countries where steel construction seems to have retained its competitive capacity.

Most of the ideas that have here been put before you are not new or anywhere near it. Others too might have been brought forward. Here I am thinking especially of the growing importance of the dynamic effects on light structures; the possibility of a reduction of the factor of safety against buckling if allowance is made for the diaphragm action of stiffened members; the utility of tubular stiffeners on plate-webbed girders; the advantages that might be gained from a thorough study of the sequence of runs in welded work; the possibility of controlling residual stresses or, still better, of turning them to good account by making them serve for prestressing; the benefit that might accrue from a careful study of the erection of structures; the possibilities offered by a suitable infrastructure, even to the point of making good use of active pressure in the soil; endeavours to lighten the many bracing members that are so often overdimensioned; the value of lattice girders with three chords, etc.

No doubt, some of the ideas mentioned call for research; this is so for plastic calculation, to be developed jointly in conjunction with welding, and the application of probability theories to the estimating of live loads and to the design of steel structures.

Constructional steelwork is the most traditional of all modern building methods including timber, and it is imprisoned in a straitjacket of regulations sometimes equally traditional. Its builders and its users have insufficient faith in its possibilities; its students, its planners and its designers are handicapped, especially as regards welded structures which are now in the majority, by apprehensions of such effects as contraction, brittle fracture, fatigue fracture, and so on.

Without being, or ever having been, professionally tempted to any discrimination or suspicion concerning modern building materials, I affirm that at the present time constructional steelwork is neither conceived nor calculated nor carried out in a rational way. Its possibilities are largely underestimated because of imaginary fears and the almost total lack of numerical data for them.

Too much work is done independently : even in one and the same country the many builders concerned in tendering for a projected work study it in the light of their own resources and, overburdened with a thousand daily cares, instead of concerting the efforts of all or of a chosen few in order to explore thoroughly one or more of the solutions put forward and perfecting these.

Steelmakers and designers ought to arrange experiments—these need not be numerous—on actual structures, tests to be carried to destruction after careful investigation and appropriate choice, to arrive at the relation between breaking loads and actual loads.

Very many identical structures will have to be put up in connection with the construction of motorways and elevated roads in towns, for which purpose it is necessary to agree upon modules and arrive at a certain degree of standardization without passing over into monotony.

Finally, and this is essential, we must at all cost escape from the beaten tracks left behind by riveted construction and its rolled sections, almost a century old, to look for new forms of construction and new sections. In this context box or tubular design seems to me a road with a future. To become convinced of this, one need merely glance at the breathtaking success achieved along this road by aircraft construction.

Friedrich W. BORNSCHEUER

Contribution to the Discussion on Model Statics

(Original text: German)

Prof. Louis referred in his report to the value of model tests. Unfortunately, model analysis is still far too little used, especially in connection with structural steelwork, so that, as Prof. Louis points out, we are depriving ourselves of the possibility of testing new concepts and new ideas. Model tests alone can help with the present abundance of shapes which architects are offering to engineers. For plane frames it is frequenthy quite impossible to carry out static calculations in their traditional form, since the analytical and numerical methods allow only of calculating the simplest load-carrying members.

Model statics has developed in recent years into an independent branch of structural analysis. One of the oldest and best known methods is that of two-dimensional stress optics which, supplemented by cross-elongation measurements with a lateral extensometer is the standard procedure for determining the stress condition in discs.

The moment distribution in plates of constant thickness can be fairly easily determined by simply using curvature measuring apparatus from which the influence surface can be drawn directly. The most important method of solving almost all problems in model analysis is the determination of the stress distribution with electrical strain-gauges. Suitable plastics are preferred for use as the model material. Because of the large number of measurements involved in model analysis, the whole procedure, including the loading and unloading of the model, is to a great extent automated. An appliance for connecting 200 strain-gauges is used in the "Institute of Stress Optics and Model Analysis" of the Stuttgart Technical College. The Institute is quite prepared to furnish details of its construction to anyone interested. The test values are punched out on tape and can be used for further calculations on digital computers.

In this way evaluated results for several cases of loading can be made available in tabular form in the shortest possible time.

And now a few supplementary remarks. While the reinforced concrete industry is already making good use of the advantages that model analysis offers, the structural steel industry lags a long way behind. Maybe this is due to the cross sectional shapes of the beams. The sectional shapes of reinforced concrete members are simpler and make for somewhat easier copying in plastic. But this is a fallacy. In my opinion, it is hardly more difficult to idealize the external isotropy of resolved steel structural members in a model than the internal, that is the material isotropy of the reinforced concrete, in which Stage II with cracked tension zone, is not even considered.

In this connection I would call to mind the introduction of the orthotropic plate in structural work. In those days, Dr. Cornelius, had used the material orthotropy of the Huber differential equation, which corresponds with the reinforced concrete member, for the calculation of the external orthotropy of narrow-meshed steel grids. Why then should a similar condition not apply to model analysis ? Naturally certain conversions will be necessary in order to obtain the stress condition in the distributed system.

It is in the nature of things with model analysis experiments that a not inconsiderable expenditure on apparatus and measuring equipment, as well as on trained personnel, is involved. This means that it is not economic for individual firms and offices to run their own special laboratories. But what is there against setting up suitably located model analysis laboratories on the lines of the electronic computing centres ?

J H. VAN DER VEEN

Duality in the Classification of the Strength of Rolled Products

(Original text: French)

This contribution is not intended to exhaust the subject, but merely to bring forward a point of discussion which is rather important in connection with the production, application and standardization of rolled products.

Originally, the strength of rolled products was fixed in terms of the tensile strength. This value is determined by a tensile test in which the steel is stressed in uni-axial tension, which produces an elongation which is first of all elastic, but becomes plastic before fracture occurs.

The tensile strength is defined as the load obtained in the test when instability or local reduction of section commences, divided by the original section area of the test piece. This value is not to be confused with the true strength when the steel breaks. The latter value is about twice as great.

Tensile strength is not a purely physical property of steel. It is influenced by the geometry of the test piece. When the width of the test piece is very great compared with its thickness, higher values are obtained than with a normal test piece. For machined test pieces the values become even higher.

Although the precise significance of tensile strength in the calculation of structures is doubtful, the classification of steel qualities has been based on this value. For a long time this basis has been used to distinguish between steel 37, with a tensile strength between 37 and 45 kg.sq.mm, and steel 42, with a tensile strength between 42 and 50 kg.sq.mm.

Later, as a further basis for design calculation, structural engineers began to interest themselves in the yield point, another property which could be determined during the tensile test. The yield point is reached at the stage when plastic deformation of the test piece commences.

Assuming that plastic deformation is not allowed in structures anywhere, the yield point seems to be more justified as a basis for calculations than the tensile strength.

Therefore, although the classification of steel quality based on tensile strength was still retained, it became the practice to include in Standards minimum values for the yield point. These values were adopted to those which could normally be guaranteed for the existing steel qualities. In this manner, for example, the minimum yield stress for steel 37 was fixed at 24 kg.sq.mm.

At first, this duality did not raise any serious difficulties, probably because the ratio between yield point and tensile strength was the same for the various products. Nevertheless, it was found that a definite minimum value for the yield point could not be guaranteed for all the various thicknesses.

This was the consequence of the metallurgical fact that the ratio between yield point and tensile strength decreases as the thickness of the product increases. This has led to a range of minimum values for the yield point as a function of thickness.

For steel 37, for example, the minimum yield point is 24 kg. sq.mm. for thicknesses up to 16 mm. 23 kg.sq.mm. for thicknesses from 16 to 40 mm. and 22 kg.sq.mm. for thicknesses between 40 and 63 mm.

Subsequently, other complications arose. In recent years, steels in general have become more pure. The content of residual elements such as copper, nickel, etc., has diminished. This also applies to nitrogen and phosphorus.

In addition, developments in the rolling mills have led to higher rolling temperatures for the ordinary qualities. These factors have led to a reduction in the ratio between the yield point and the tensile strength. The values stipulated for the yield point can always be guaranteed, but for the ordinary steels the minimum tensile strength is higher than before.

Furthermore, the introduction of welding, with the associated problem of brittle fracture, has led to the development of special steels which differ from ordinary steels by their higher resistance to brittle fracture, often at low temperatures.

These types of steel are now well known, often used and have found their way into Standards. In their manufacture, the producers generally employ methods which lead to a very fine ferritic grain and, in addition, they use for example, an increase in the ratio between manganese and carbon, or they add various alloying elements. All these factors taken together lead to an increase in the ratio between yield point and tensile strength. This means, that for steels with the same degree of tensile strength it is possible to increase the guaranteed minimum yield point, to the benefit of the fabricators, or that for the same yield point the tensile strength should be reduced For thicknesses between 25 and 40 mm., for example, ordinary quality plates now show a ratio between yield point and tensile strength of about 0.54, while for plates of superior quality the ratio reaches 0.66.

This means that to guarantee the minimum yield point of 23 kg./sq.mm. (in the case of steel 37) the value of the tensile strength should be 42 to 50 kg./sq.mm. (equivalent to the American steels A7 or A283, Grade C.) By contrast, for the higher quality a minimum tensile strength of 35 kg./sq.mm. would be adequate.

Another complication in the classification of steels results from the fact that there is a difference between rolled sections and plates. For plates, the ratio between yield point and tensile strength is normally smaller than for sections owing to the influence of the cooling speed. For a same quality steel 37, for example, with thicknesses between 16 and 40 mm., the values of the minimum yield point could be 21 for plates and 23 for sections. However, the significance of the yield point to structural engineers appears to be of such importance that the fabricators on standardization committees demand the same yield point for plates and sections. This means that in actual fact the range of tensile strength of plates of ordinary quality must be changed from 37-45 to 42-50. These examples show that the importance attached to the yield point while retaining the old classification based on tensile strength has led to a great number of difficulties and anomalies. It seems necessary for us to choose which is the principal criterion for fabricators. If the yield point is selected, it is quite possible that a new classification, clearly based on this value, will result in less complication and will be more acceptable to fabricators.

Leo FINZI

Design Studies for Wheel-Plan Tension Structures

(Original text: Italian)

The use of cables for large roof constructions spaces has in the past decade, resulted in a large number of important developments, but, as on previous occasions in history, technique has partially outstripped science as regards a rigid theoretical and experimental study of the subject.

Although the use of a double frame of load-bearing and tension cables (a "new idea" which has opened the way for future outstanding designs) is now the well-established basis for a large number of interesting forms, so far there have been few studies designed to provide simple methods of calculation whilst offering a guaranteed degree of approximation.

We are faced with a relatively small developed field of constructional science which, whilst admitting the elasticity of the material, studies the non-linear consequences of loads in structures which are highly prone to deformation.

Secondary effects, well known to engineers and builders of suspension-bridges, play a primary role in tension structures. The relevant literature is very recent and only covers a portion of the wide field involved. Special attention should be devoted to the effect of typical asymmetrical loads such as wind forces, thermal co-action, and viscoplastic give, in members and elastic and aero-elastic stability.

I found this lack of scientific and technical documentation very noticeable in designing the 68 m. diameter central glass dome of the Genoa Palasport (1 and 2) with Professor Maier. This is a wheel-plan tension structure with a central nucleus consisting of a hyperbolcid lattice grid (3). The wind-tunnel distribution of wind forces (Fig. 1) clearly reveals the complete asymmetry of the incident load. This was dealt with by developing a fairly simple and approximate method of calculation.

Another major problem studied was the instabiltiy of the compressed perimeter ring. With protection from radial cables in its own plane and from elastically yielding vertical supports in the perpendicular plane, its elastic equilibrium was considerably stabilized.

As part of the same design I investigated the relationship between the shape of the perimeter ring and the prestresses to be distributed amongst the cables in order to ensure optimum conditions.

A final feature which may be of interest is the theoretical and experimental study (Fig. 2) of the central part from the point of view of overall and local stability.

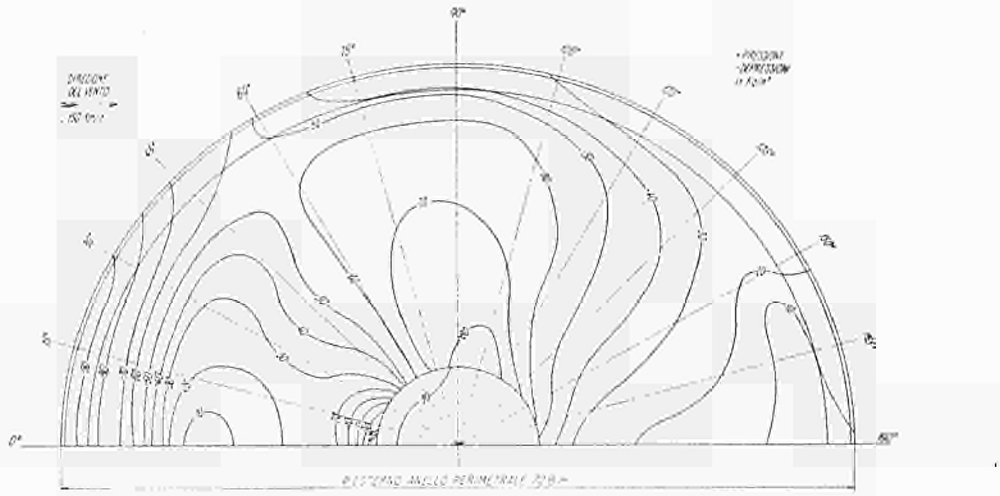

Fig. 1

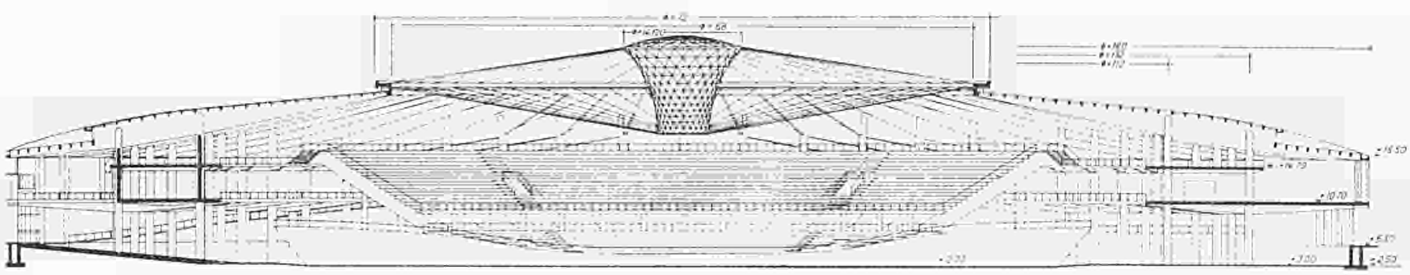

Fig. 2

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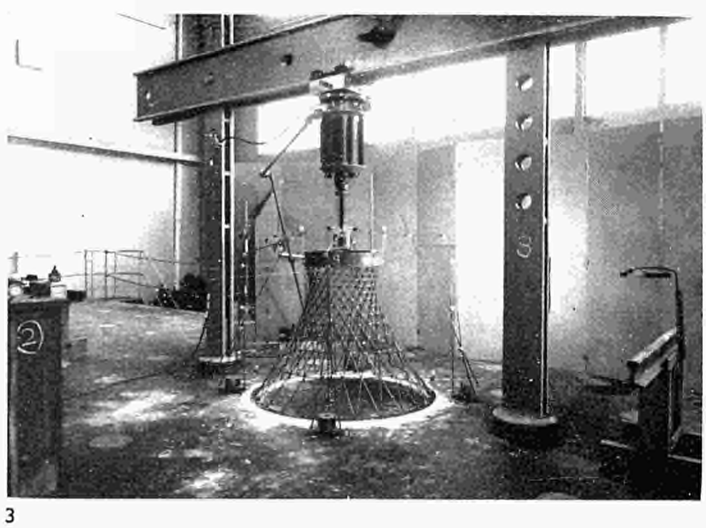


3

Z. S. MAKOWSKI

Analytical and Experimental Investigations of Stress Distribution in Steel Space Frames

Original text: English)

In the course of this Congress many references have been made to the recent developments in three-dimensional structures and various examples of steel space frames have been shown by Professor Beer in his most interesting lecture.

If we survey this field in detail, we will be impressed by the obvious impact which space frames are now exerting upon modern architecture and the great variety of new forms in which three-dimensional structures have been used during the last few years. Many progressive designers, realizing the advantages of space frames, are now taking a keen interest in this form of construction.

In nearly all buildings there is now a general tendency to reduce the number of intermediate columns and as a result there is a trend towards large span structures.

There is also a great emphasis put on prefabrication and mass-production in the factory.

The introduction of industrialized systems is an answer to the demand for increased speed of construction, and a possible reduction in cost.

All these requirements are satisfied by modern space structures. They are ideally suited for covering exhibition halls, assembly rooms, churches, swimming pools and industrial buildings, in which large unobstructed areas are required.

The recent emphasis on prefabrication has drawn the attention of designers to the fact that space frames can be built up from simple, prefabricated units, in many cases of standard size and shape.

Such units, mass-produced in the factory, can be easily and rapidly assembled on site by semi-skilled labour. At the same time the small size of the units greatly simplifies the handling, transportation and erection, as no heavy hoisting equipment is required on the site.

Most space structures have been built in steel; it is a strong, reliable, adaptable and highly economical material, especially for large spans.

In addition to pleasing architectural appearance, space structures offer a number of distinct advantages—great stiffness and rigidity, uniform stress distribution, even under unsymmetrical loading.

Concentrated loads can be accommodated in space frames more easily than in simpler conventional forms of structure due to the omni-directional distribution of stress.

Space frames have a built-in reserve of strength enabling a structure to take local overloading. Past experience shows that space structures, even when badly damaged, never collapse rapidly; this phenomenon is of great importance in case of fire.

Having greater rigidity, the space frames allow also greater flexibility in layout and certain changes in the positioning of columns as it is possible for some of them to be removed or have their position modified without damaging the structural integrity of the framework.

In the past the chief barrier preventing greater use of three-dimensional structures was the complexity of design calculations.

As a rule, space structures are highly statically indeterminate and their analysis by exact methods leads to extremely tedious computation if attempted by hand. This has been the reason why approximate methods of analysis have been used in the design of three-dimensional structures, but in using approximate methods the danger of under-estimating stresses can only be avoided by over-estimation, and hence many space structures that have been built have been designed with an unduly high factor of safety, and as a result the economical advantage of good design has been lost.

The use of the electronic computer has radically changed the whole approach to the analysis of space frames. Instead of using approximate methods, we are now returning back to precise methods of calculation.

With the aid of an electronic computer, it is now possible for the engineer to analyse very complex space structures with much greater accuracy than ever before and with a marked reduction in the time involved.

A great deal of fundamental work on three-dimensional structures has been done at the Space Structures Research Centre of the Department of Civil Engineering at Battersea College of Technology in London. At present the Research Centre consists of two members of the academic staff and seven full-time research students working for higher degrees of the University of London.

The Centre carries out the analytical and experimenta investigations into stress distribution in various types of space structures and prepares general programmes for electronic computers.

The result of this work is prepared also in the form of graphs and tables so as to be of direct use to practising engineers interested in the design of space frames.

The policy of the Centre is to preserve a balance between pure and applied research. A very close contact is maintained between the College and industrial firms in various countries, who specialize in the production of space frames.

At present research is concentrated on the following topics:

(a) single and double-layer grids

(b) braced barrel vaults

(c) braced domes

(d) transmission towers.

During the investigations of stress distribution in these structures, full use is being made of the College electronic computer. The computing facilities consist now of the Ferranti Sirius Computer and will be improved further by the installation in the near future of an Elliott 503 Computer.

In addition to analytical studies, experiments on models of space structures are being carried out in the College Research Laboratories, sometimes as a check on the accuracy of the analytical approach, sometimes as the only means of finding the stress distribution in very complex structures of unorthodox layout, for which the mathematical approach has not been fully formulated.

Various types of flat single-layer grids (Fig. 1) have been studied in detail. The stress distribution in these structures depends to a large extent on the boundary conditions and type of loading.

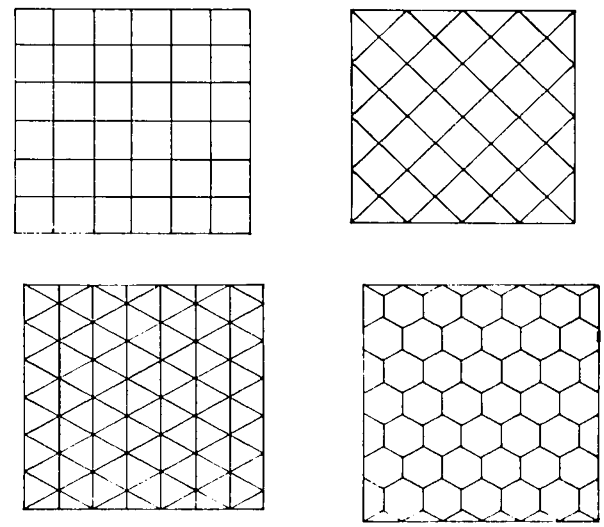


Fig. 1

For grids of this type, Mr. H. Nooshin, working under the supervision of the author, has produced a general programme for the computer based on the slope-deflection method using matrices.

In this approach, the forces acting upon each of the members of the grid are given in terms of fixed end moments and reactive forces and displacement of both ends of the members.

Using the equilibrium conditions at all joints of the grid, linear simultaneous equations can be set up, which after solution give the displacement components of all the joints. Back substitution into the original slope-deflection equations yields the generalized forces in all members of the grid.

This approach, although too tedious to be practical for hand computation with complex grids, can be applied with advautage to even very complicated grid structures, if use is made of an electronic digital computer.

In Mr. Nooshin's programme prepared for the Sirius Autocode, full account has been taken not only of the stress resultant produced by bending and torsional moments, but also of the shear strain energy.

Using matrices, the conditions of equilibrium of the structure can be expressed as:

$$Kd = W$$

where K = stiffness matrix for the whole structure,

d = vector of all unknown displacements,

W = vector of all fixend and reactive forces.

Using the general programme, the stress distribution of different grid layouts as well as "open"-and-"closed" type grids, pin-jointed, rigidly connected with or without edge beams, simply supported, encastré or supported on columns have been fully investigated, and the effects of torsion and shear deformation have been studied in detail.

The general conclusion reached is that in grids having rigidly connected members of rectangular cross-sections the effect of torsional rigidity is important, whereas the effect of shear energy is negligible—on the other hand the reverse is true for steel I-sections.

Using the general programme, hundreds of grids have been analysed for varying numbers of subdivisions of the network.

Figure 2 shows a typical case of a diagonal grid having the subdivisions 6 × 6, supported at four corners, under the action of uniformly distributed loading covering the whole area.

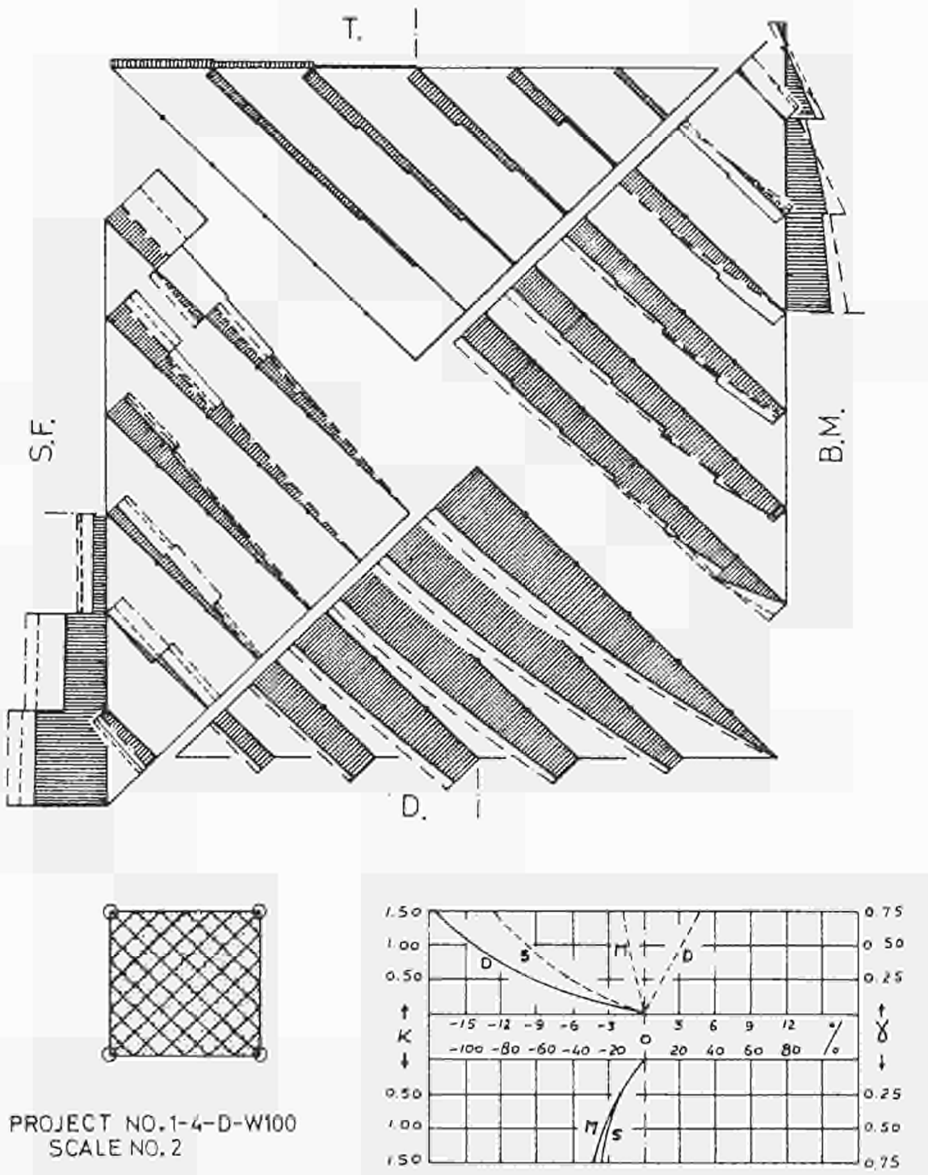


Fig. 2

Figure 3 shows an identical grid but simply supported along all four sides. Figure 4 represents an encastré grid.

The diagrams show the distribution of bending and torsional moments, shearing forces and deflections, obtained under three different assumptions:

(a) when the strain energy due to torsion and shear is neglected (—·—·—·—);

(b) when the strain energy due to torsion is considered but that due to shear is disregarded (————————);

(c) when the strain energy due to shear is considered but that due to torsion is neglected (----------).

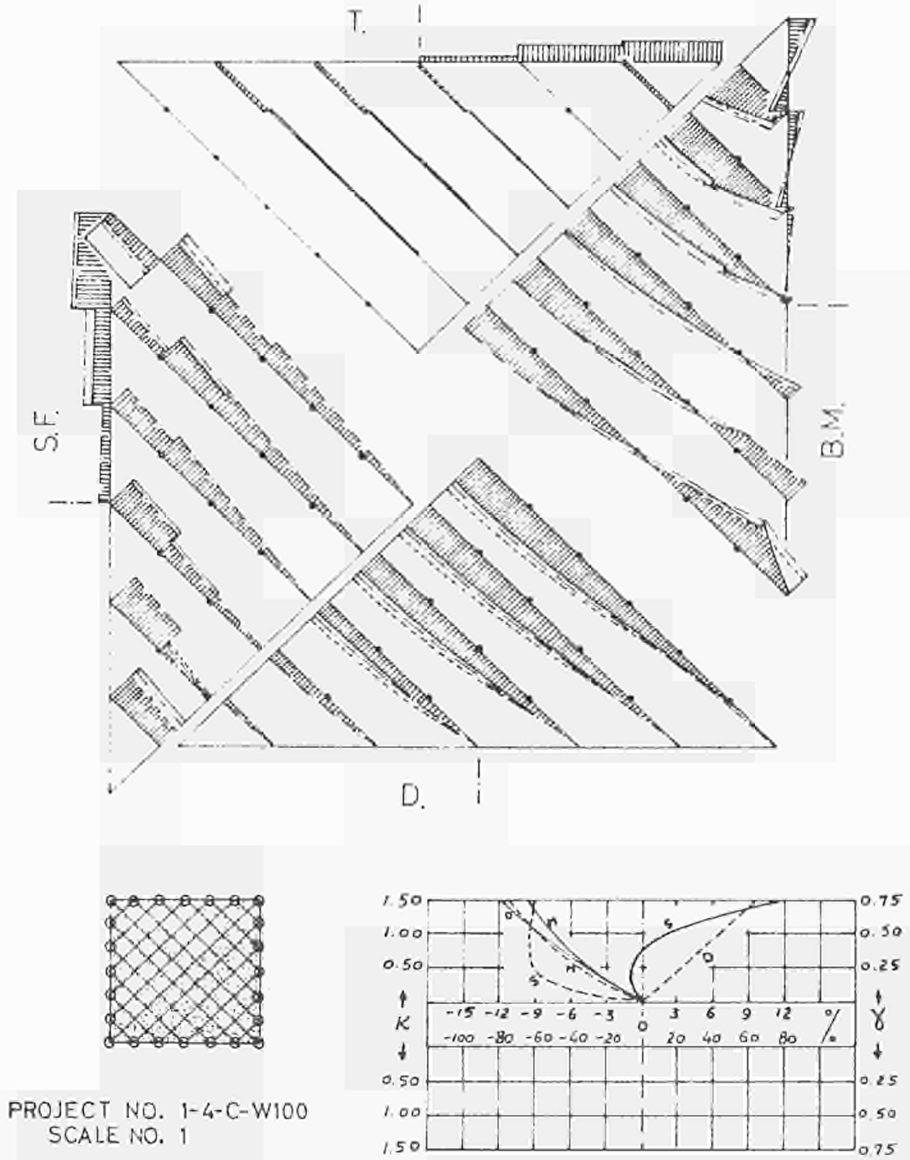


Fig. 3

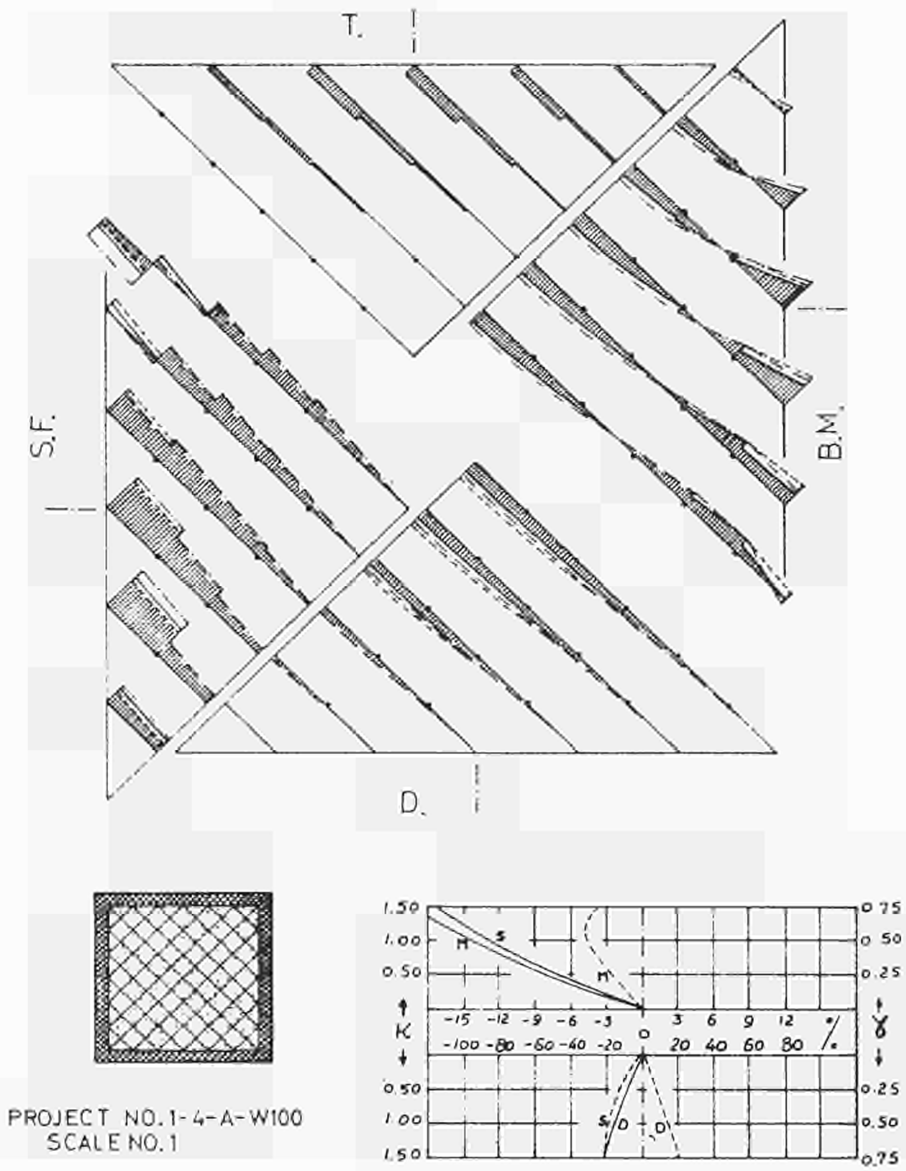


Fig. 4

In recent years, the Space Structures Research Centre has carried out a number of structural stress analyses for steel space grid structures, on behalf of contractors and consulting engineers responsible for the design.

Figure 5 shows the layout of a three-way steel latticed grid which will cover the King Hussein's Sports Stadium in Jordan. Full structural analysis has been carried out on behalf of Booth (Steel Structures) Ltd. including the determination of axial forces in all members of the structure, bending moments, shears and deflections.

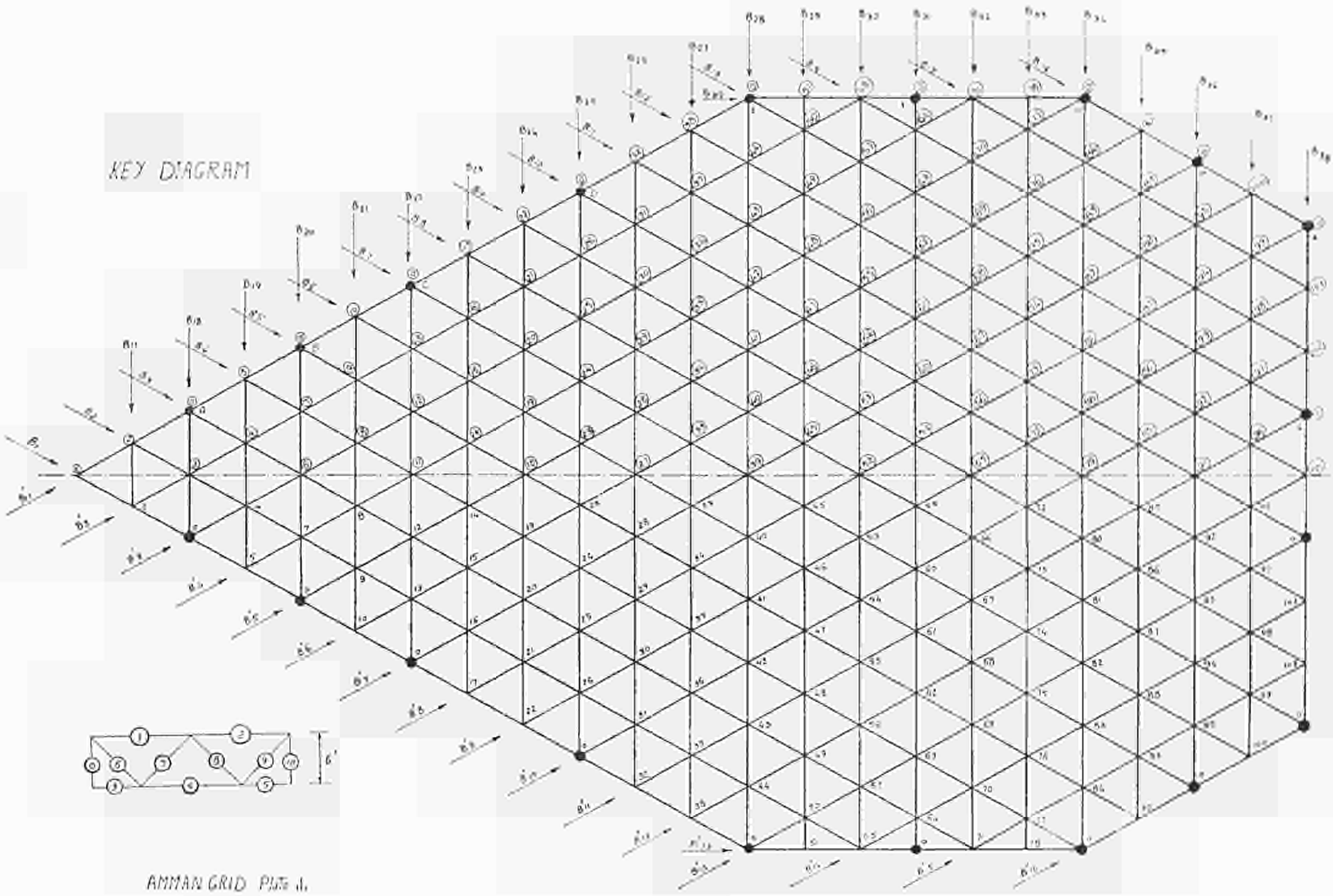


Fig. 5

Acting on behalf of the same firm, the author has prepared a number of designs for three-way double-layer grid structures, all of them built in steel. In Great Britain this type is known as the Met-Ram construction and consists of prefabricated latticed steel units which can be interconnected speedily on the site into a three-dimensional grid.

The analytical investigations of stress distribution in this form of construction have been supported by the testing of a full-size prototype, erected at Feltham, near London (1).

The span of this "model" was 56ft. The tests carried out under the guidance of the author confirmed the great reserve of strength of this type of space structure.

Several assembly halls, industrial buildings and large-span canopies have been built in steel using this system.

Figures 6 and 7 show a plan of the recently built National Exhibition at Nancy in France. The Exhibition is housed in five large-span pavilions covering an area of over 183,000 sq.ft. Three identical halls of square layout have a clear span of 149 ft., one double hall measures 149ft. by 298ft. and the largest hall covers an area of 149ft. by 468ft. without any internal columns. All these pavilions have been built as double-layer grids and consist of steel prefabricated pyramidal units, all of the same dimensions. The units are made of angle sections connected by welding and are mass-produced in a jig in the factory and connected together at the site with bolts.

Some 900 tons of galvanized steel have been used in the construction of these structures. The double-layer two-way space grids, arranged diagonally for greater rigidity, have proved to be highly competitive in comparison with conventional roof trusses. Erection was extremely simple; all the roofs were assembled at ground level and hoisted up bodily, using erection towes.

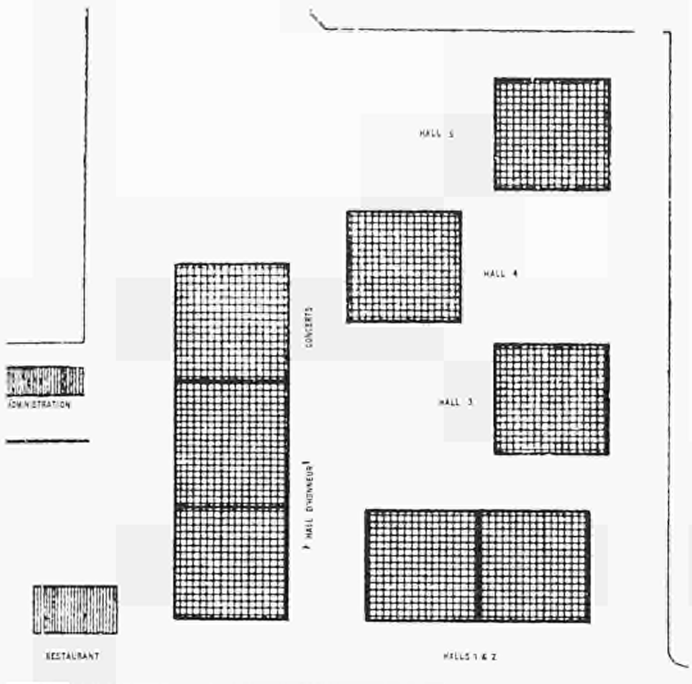


Fig. 6

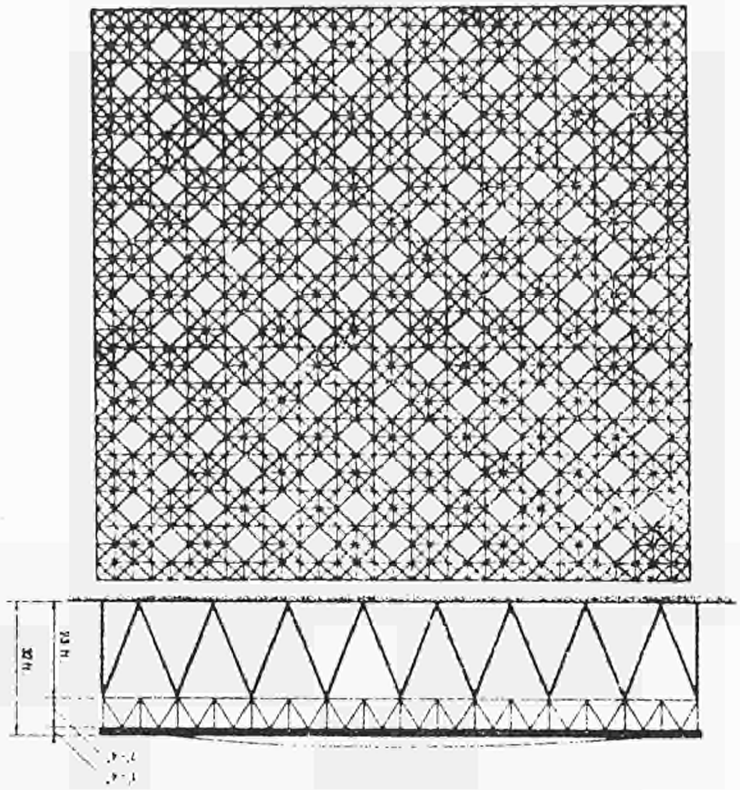


Fig. 7

At the request of the French consulting engineer for this structure, Mr. S. Du Château, the analysis of these highly statically indeterminate structures has been carried out by the author and Mr. H. Nooshin at the Space Structures Research Centre at Battersea College of Technology using a specially developed programme for the electronic computer.

The author has had the opportunity to act on several occasions as a consultant to Mr. S. Du Château on the structural analysis of steel space structures built recently in France by this famous exponent of steel space frames.

Figure 8 shows the diagram of bending moments for the simplified network of the three-way grid covering the swimming pool, Piscine Du Stade Francais at Bilancourt, Paris, produced by a uniformly distributed load on the whole area as determined by the author with the use of the electronic computer. This structure was built according to Mr. S. Du Château s design in 1961-62.

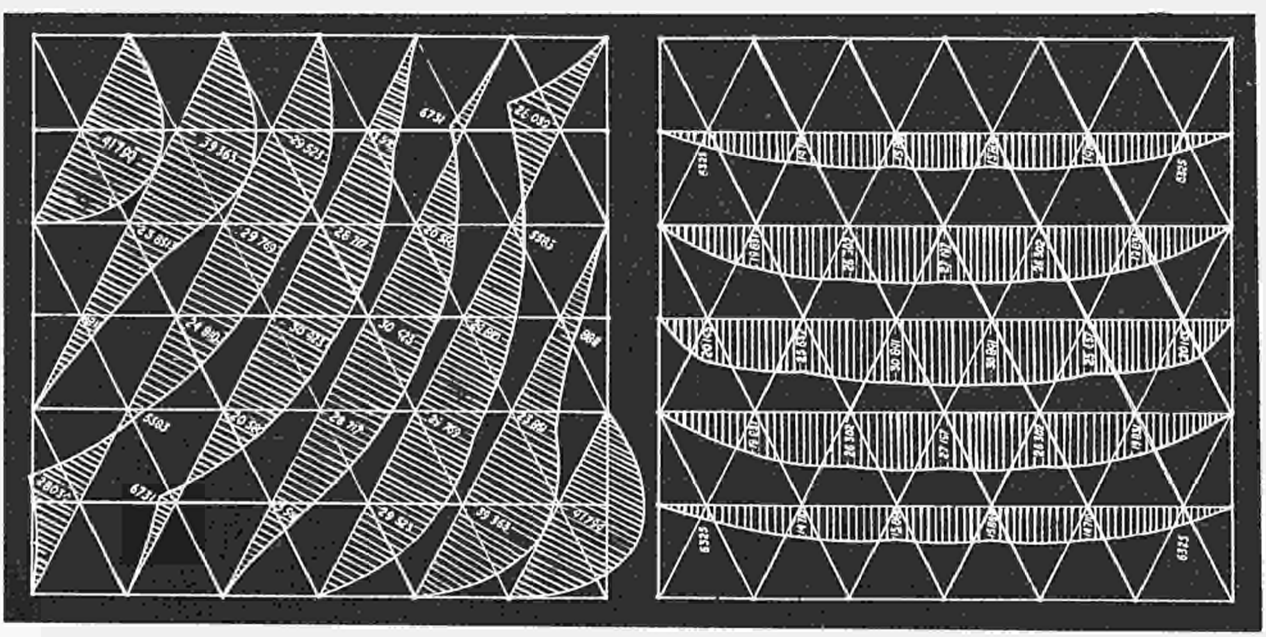


Fig. 8

Mr. S. Du Château has also introduced into France a prefabricated double-layer grid system, known as "Pyramitec." This system has proved to be highly economical and has been used with great success for many industrial buildings.

The Space Structures Research Centre at Battersea College has carried out on behalf of Mr. S. Du Château a number of investigations into the stress distribution in the Pyramitec system.

Figures 9 a, b and c show the results of a stress analysis for a grid built in 1964 in France covering a factory 35m. × 35m.

Several prefabricated steel grid systems are now commercially available in various countries. One should also mention perhaps the American Unistrut system or the Space Deck system which has frequently been used in the U.K. The structural advantages of the Space Deck system have been realized by the British Ministry of Public Building and Works. Their Directorate General of Research and Development introduced last year the Nenk system of prefabricated steel and construction based on a modification of the Space Deck system using it not only for roofs but also as floor decks in multi-storey barrack block structures built for the War Office. In the Nenk system the double-layer two-way grids are formed from prefabricated steel pyramids units 4ft. square in plan and only 2ft. deep.

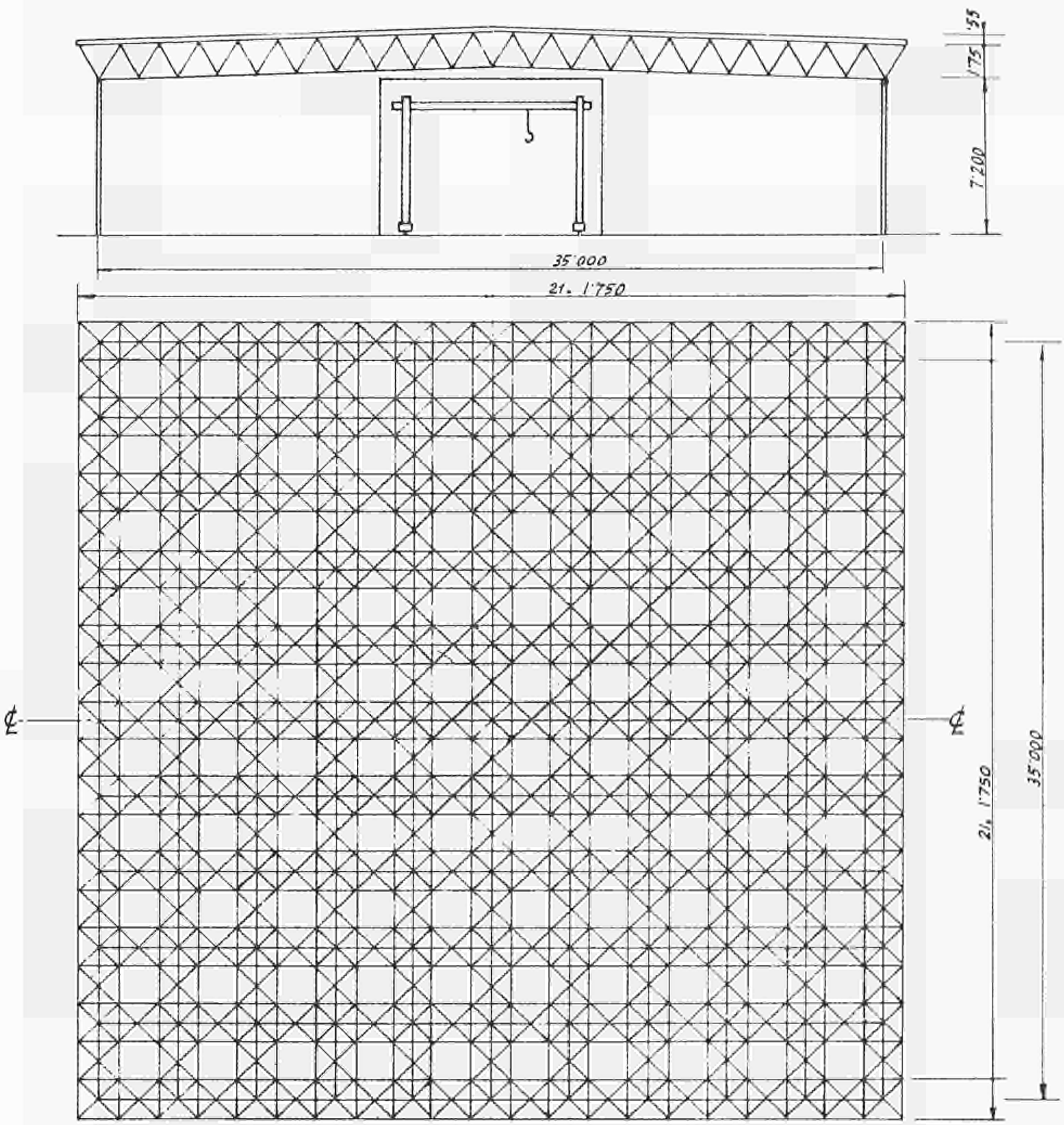
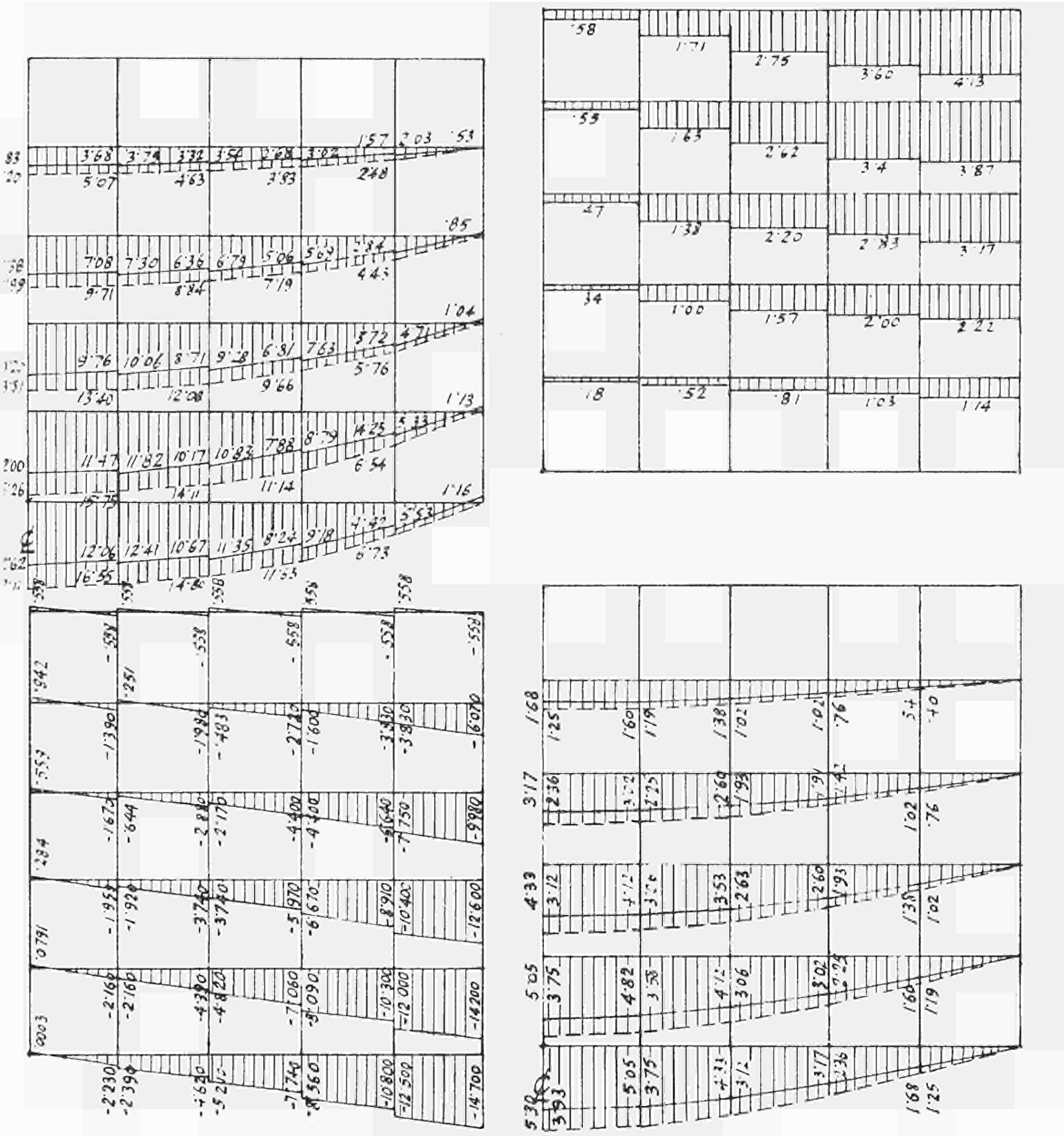


Fig. 9a



·58
1·71
2·75
3·60
4·13
·55
1·63
2·62
3·4
3·87
·47
1·38
2·20
2·83
3·17
·34
1·00
1·57
2·00
2·22
·18
·52
·81
1·03
1·14

Fig. 9b

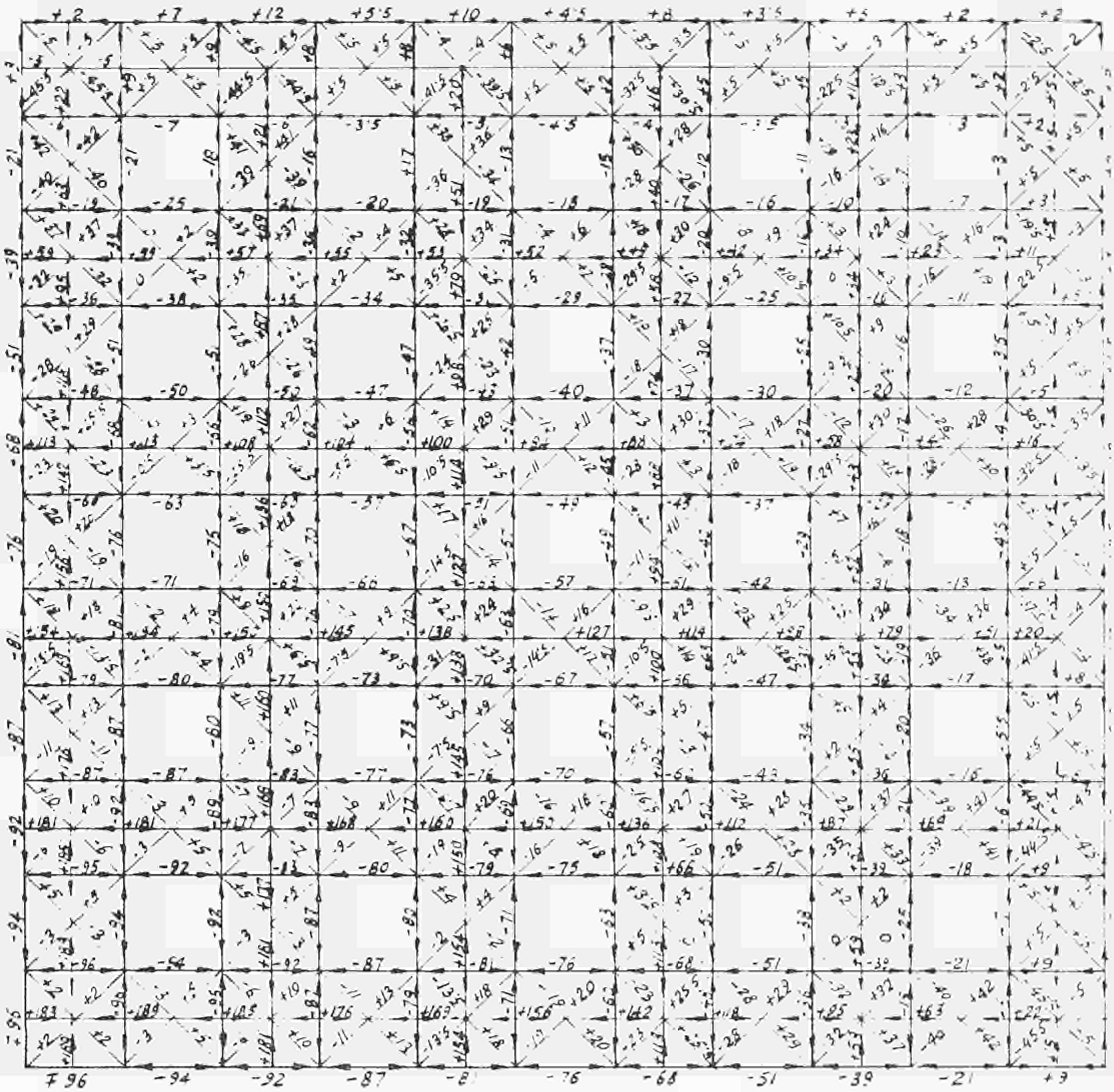


Fig. 9c

Photograph 2 illustrates the erection of a building in which the Nenk system is used for support of the floors.

In 1964 the Building Research Station at Garston, Watford, carried out extensive tests on a full-size multi-storey structure incorporating the Nenk system which proved to have great rigidity.

Braced barrel vaults form another type of space structure which now receives special attention at the Space Structures Research Centre at Battersea College. A braced barrel vault in a structure similar in configuration to a shell; however, it is not homogeneous, being an assembly of bars.

Different types of bracing have been investigated and their relative stiffnesses determined. Some of the small-scale models, on which tests have been carried out, are shown in photograph 3. All models have the same length, width and rise and their bracing members were arranged in such a way that the total weight of the material used to construct the models was almost the same in each case. The tests on the models showed significant differences in the behaviour of the structures proving that the type of bracing has a fundamental influence upon the strength and load-carrying capacity of a braced barrel vault.

Figures 10 a, b, c and d show the deformation of some of these models under uniformly distributed loading covering the whole area.

One of the research students in the Department of Civil Engineering at Battersea College, Mr. S.V. Velankar, is working at present on the determination of stresses in barrel vaults having the three-way type of bracing. Various methods of analysis have been compared and a precise solution obtained by means of a general electronic computer programme.

The equilibrium approach has been used. The advantage of this method is that in developing equations we are concerned with single members only and their elastic properties. The arrangement of the bars, or the topology of the structure, can be taken into account when preparing the data for the computer.

As the storage capacity of the Ferranti Sirius computer, for which the programme has been prepared, is limited to only 7,000 words, the computation programme is divided into three parts.

In the first part, the actual structure is divided into a certain number of groups of joints such that the joints in the i^{th} group are not connected to any other group except $(i-1)^{th}$ and $(i+1)^{th}$.

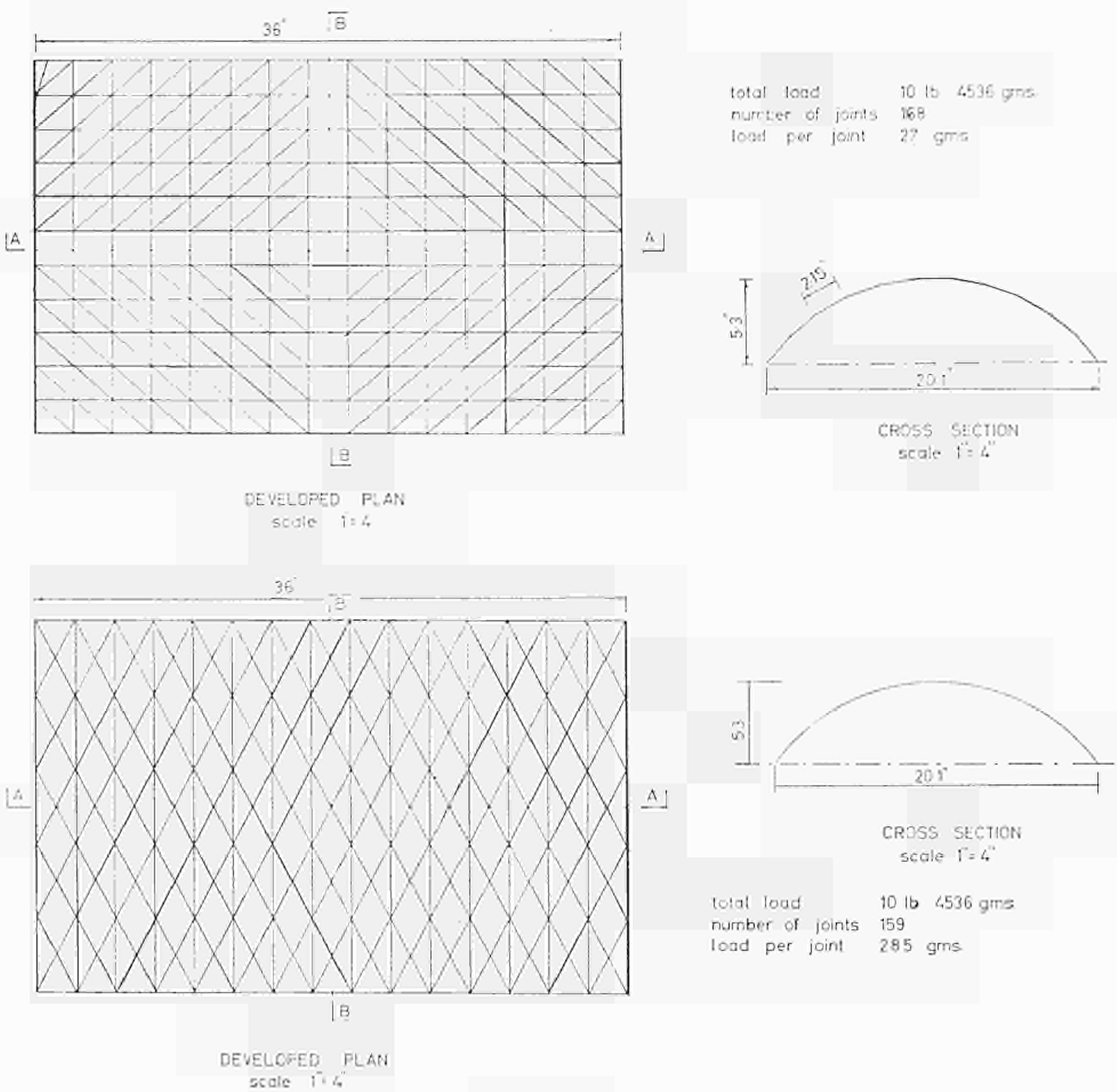


Fig. 10a

Deformations of the i^{th} group are then expressed in terms of the preceding group $(i - 1)^{th}$.

The process of expressing the deformations is continued until finally the actual deformations of the last group of joints, not being connected to any further groups, can be obtained.

Having obtained the relationship between the deformations of all the nodes and the actual values of deformations of the last group, the computer, by back substitution calculates the deformations of all the joints of the structure. This forms the second part of the programme.

The third part consists of the determination of the actual axial forces in all members of the structure from the known components of displacements of the ends of the members.

The programme, which was originally prepared for pin-connected structures only, is now being extended to rigidly jointed systems so that in addition to axial forces, it will be possible to obtain also bending and torsional moments. The programme is greatly simplified if the barrel vault consists of tubular members of circular cross-section.

Having a general programme it is possible to compare the forces obtained for various rise to span ratios. This has been done in figure 11.

It is assumed that the structure is subjected to loads applied to all unit concentrated internal nodes, and that the barrel vault is supported only along rigid diaphragms at each end and has only vertical movement permitted along the edge longitudinal booms.

The axial forces shown in the diagrams illustrate how change in the rise to the span ratio affects the stress distribution in the barrel.

Photograph 4 shows a model of a barrel vault which has been used for testing of the structure under various boundary conditions.

Photograph 5 shows another model of a tubular steel structure, designed by Messrs. Harris & Sutherland, consulting engineers of London.

The stress analysis of this pin-connected structure has been carried out by the Space Structures Research Centre. The model has been used to determine the effect of rigid joints on the reduction of deflections and stresses in the bars.

Steel braced domes form another extensive area in which theory can be checked in practice. The stress analysis of these structures can be carried out by a number of methods. For the triangulated types of bracing it is normally assumed that the members are pin-connected at the nodes, and that only

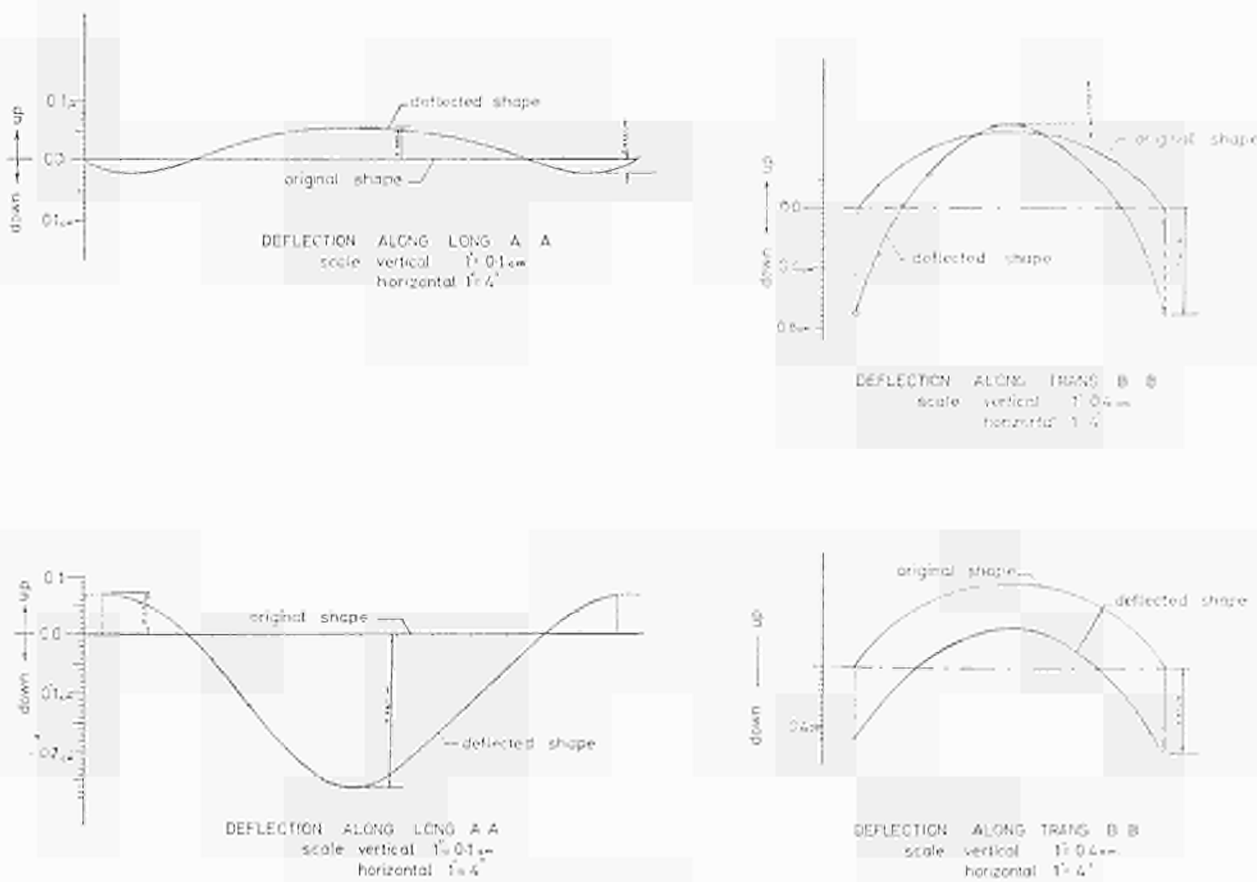


Fig. 10b

axial forces are considered in the analysis. This approximation gives quite reliable results for braced domes of high rise to span ratio with a coarse network.

For flat, low rise domes, the rigidity of connections exerts a very considerable influence, reducing considerably the deflections and modifying the loads in the members which now carry also bending and torsional moments. Certain types of bracing produce a very uniform stress pattern even under unsymmetrical loading. In the Space Structures Research Centre various types of domes have been studied analytically and also by means of small scale models. The stress distribution in three-way grid domes have been investigated in detail and special programmes developed for the electronic computer. Figure 12 shows three types for which such programmes have been prepared. Having a general programme it is very easy to get the solution for varying rise to span ratios as shown in the same figure for one of the three-way grid domes supported by rigid arches.

Photograph 6 shows the three-way steel grid dome, designed by Mr. S. Du Château and built in Chartres, France. The structural analysis of this dome has been carried out by the author. The analytical stress distribution has also been checked by the model shown in photograph 7.

The results of the mathematical analysis for the simplified version of the pin-connected dome are shown in figure 13.

The comparison of loading, cases I and II, is extremely interesting; it shows how great can be the difference in the

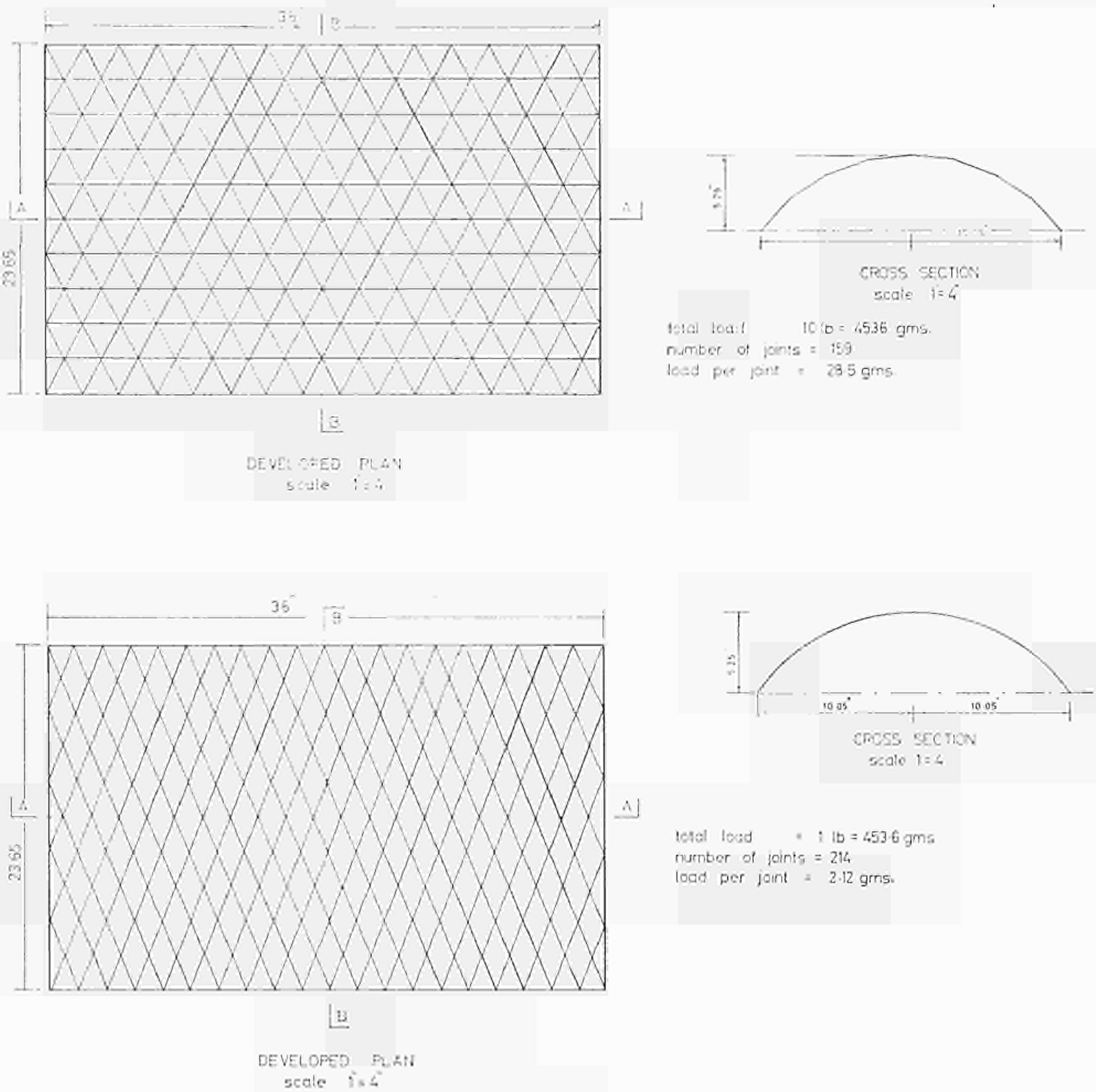


Fig. 10c

forces in the members of the dome carrying the same load, but having modified support conditions.

In case I the analysis was carried out for a dome supported along its boundary on unyielding supports A and B.

In case II the influence of yielding supports was investigated. In this case the dome is supported on vertical columns which enable vertical glazing to be provided and through which light is let into the interior of the dome. The supports of the dome are able to yield horizontally and vertically; this substantially increases the forces in the members lying near the boundary.

Cases III and IV illustrate the differences in load distribution produced by the same external loading acting on behalf of the dome only (the snow loading), but arranged differently in each case with respect to the main axis of the dome.

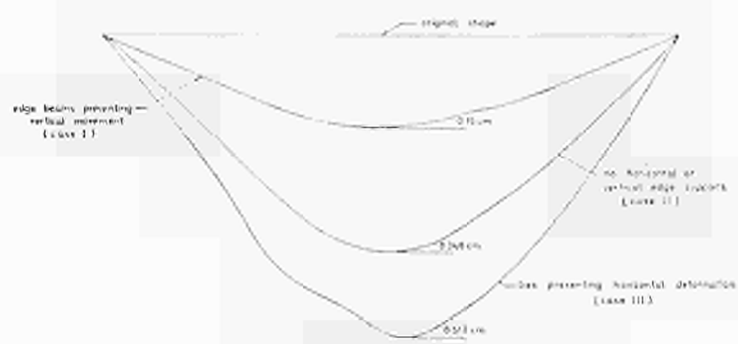
While analysing the Chartres dome, an attempt was made by the author to take into consideration the contribution provided by the bending moments in addition to the strain energy due to axial forces.

Figure 14 shows the bending moment diagrams produced by fully symmetrical loading covering the whole area of the roof and also by the condition which arises when snow covers only half of the roof.

Mr. M. J. Bayley, a research student in the Department of Civil Engineering, is investigating the validity of various simplifying assumptions used in the analysis and design of dome structures. He is supporting his analytical studies by experiments on small-scale models. Mr. Bayley is restricting his attention to domes with three-way bracing and investigating the contribution made by the primary grid members and then the influence of secondary and tertiary systems. An analysis of a dome to be built in the near future is being carried out using various assumptions. The dome has a diameter of 143 feet and will be constructed with tubular members, of rectangular section. The rise to span ratio is 1: 9.36. Flexibility and stiffness methods have been used and a computer programme prepared for various types of loading. Shell analogy is also being used as an alternative approach. A pin-connected structure was studied first and bending and torsional moments are now being taken into account for horizontal rings and ribs.

DEFLECTION ALONG LONG A A
scale vertical 1"=0.1 cm
horizontal 1"=4'

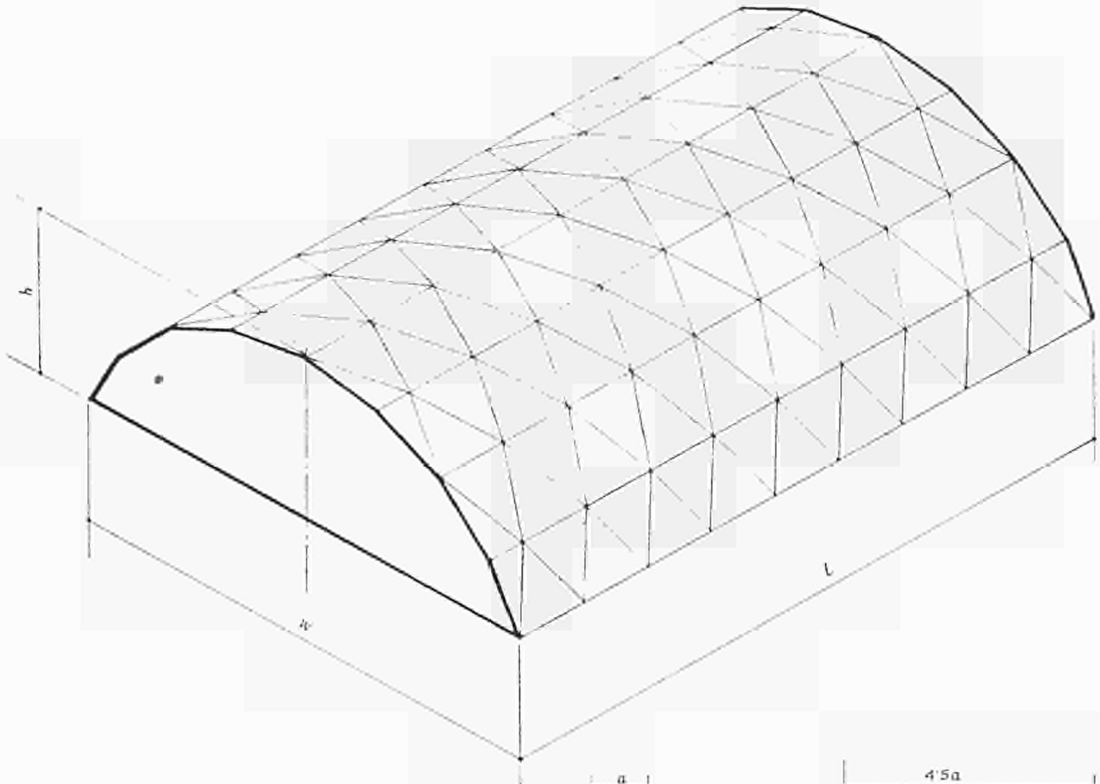
DEFLECTION ALONG TRANS B B
scale vertical 1"=0.1 cm
horizontal 1"=4'

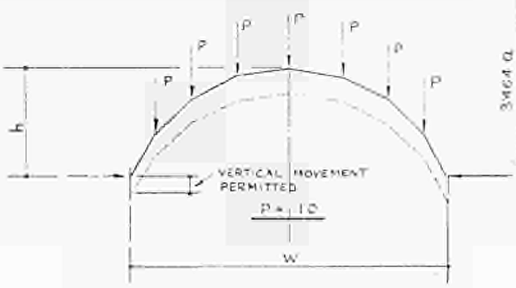


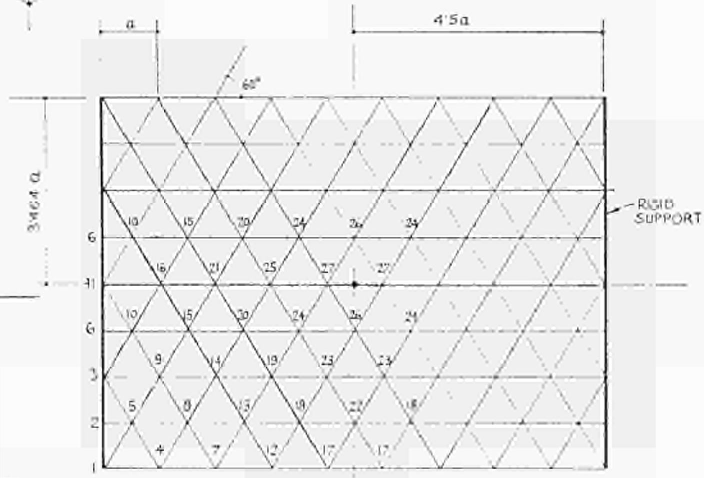
DEFLECTION ALONG LONG A A
scale vertical 1"=0.1 cm.
horizontal 1"=4'

DEFLECTION ALONG TRANS B B
scale vertical 1"=0.4 cm
horizontal 1"=4'

Fig. 10d







DEVELOPED PLAN

MEMBER	w=3h	w=4h	w=5h
1 · 4	-12·55	-19·59	-2655
1 · 5	-13·01	-18·91	-2473
2 · 5	8·54	19·57	3054
3 · 5	5·05	3·33	1·85
3 · 9	2·39	2·25	1·99
3 · 10	-5·79	-5·64	-5·71
4 · 5	10·12	14·71	1923
4 · 7	-3·14	-4·90	-6·64
4 · 8	-10·12	-14·71	-1923
5 · 8	3·41	7·82	12·21
5 · 9	-7·06	-6·79	-6·71
6 · 10	3·93	3·46	3·01
7 · 8	7·23	10·51	13·74
7 · 12	3·59	5·60	7·59
7 · 13	-7·23	-10·51	-13·74
8 · 9	3·54	2·06	0·78
8 · 13	-0·43	-0·38	-1·53

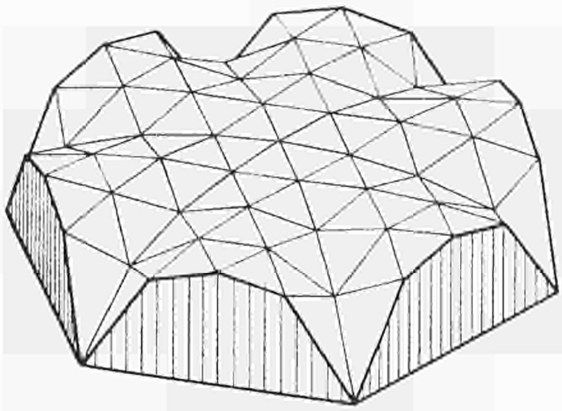
MEMBER	w=3h	w=4h	w=5h
8 · 14	-5·55	5·52	-5·64
9 · 10	1·93	0·65	-0·44
9 · 14	0·60	0·56	0·50
9 · 15	-4·82	-4·86	-5·05
10 · 11	-0·44	-1·30	-2·01
10 · 15	1·57	1·39	1·20
10 · 16	-3·09	-3·43	-3·85
11 · 16	2·87	2·49	2·15
12 · 13	4·34	6·30	8·24
12 · 17	7·62	11·89	16·12
12 · 18	-4·34	-6·30	-8·24
13 · 14	2·02	0·80	0·29
13 · 18	-2·99	-6·84	-10·69
13 · 19	-0·68	-4·25	-4·57
14 · 15	0·97	-0·13	-1·10
14 · 19	-0·68	-0·64	-0·57
14 · 20	-3·86	-4·07	-4·39

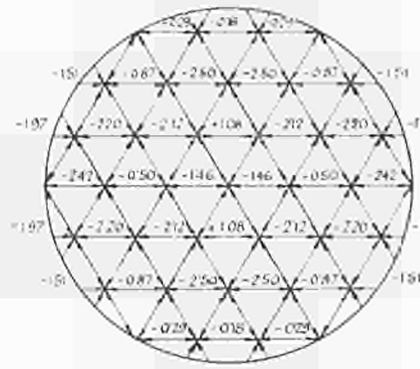
MEMBER	w=3h	w=4h	w=5h
15 · 16	-0·77	-1·56	-2·30
15 · 20	-0·20	-0·17	-0·15
15 · 21	-2·77	-3·16	-3·63
16 · 21	0·72	0·62	0·54
17 · 17	8·96	13·99	18·96
17 · 18	1·45	2·10	2·75
17 · 22	-1·45	-2·10	-2·75
18 · 19	0·51	-0·46	-1·36
18 · 22	-4·27	-9·78	-15·27
18 · 23	-2·52	-2·99	-3·50
19 · 20	0·00	-0·92	-1·76
19 · 23	-1·45	-1·35	-1·21
19 · 24	-2·89	-3·28	-3·73
20 · 21	-1·10	-1·83	-2·57

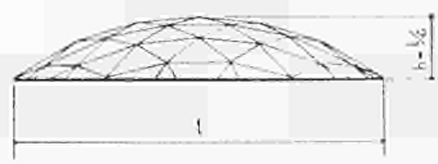
MEMBER	w=3h	w=4h	w=5h
20 · 24	-1·37	-1·21	-1·05
20 · 25	-2·42	-2·89	-3·41
21 · 25	-0·82	-0·71	-0·61
22 · 23	-1·00	-1·73	-2·47
23 · 23	-1·71	-1·59	-1·42
23 · 24	-0·96	-1·71	-2·42
23 · 26	-1·93	-2·49	-3·08
24 · 25	-1·43	-2·10	-2·74
24 · 26	-1·96	-1·73	-1·51
24 · 27	-2·09	-2·63	-3·19
25 · 27	1·74	-1·51	-1·30
26 · 27	-1·76	-2·36	-2·97
27 · 27	2·05	-1·78	-1·53

STRESS DISTRIBUTION DUE TO VERTICAL UNIT LOADS AT INTERNAL NODES

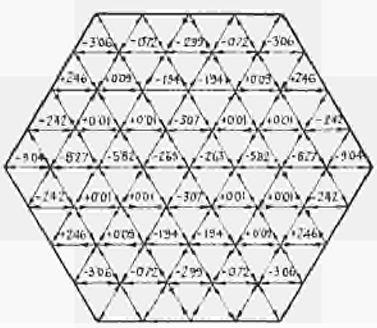
Fig. 11

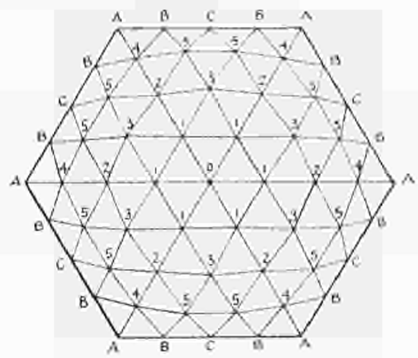


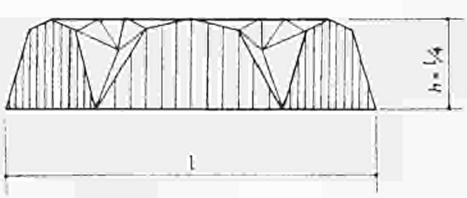




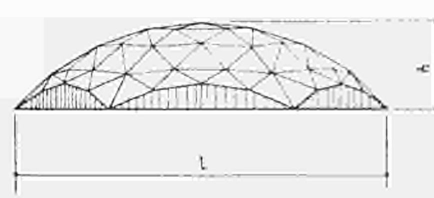
TYPE B







TYPE A



Member	0·1	1·1	1·2	1·3	2·3	2·4	2·5	3·5	4·5	4·A	4·B	5·5	5·B	5·C
l/h=2·75	-1·06	-0·89	-1·15	-1·26	-0·47	-1·44	-0·97	-1·26	0·08	-1·27	-0·55	0·10	-1·59	1·54
2·90	-1·11	-0·94	-1·19	-1·29	-0·54	-1·46	-1·02	-1·29	0·00	-1·33	-0·95	0·00	-1·60	-1·53
3·00	-1·14	-0·98	-1·21	-1·31	-0·59	-1·48	-1·05	-1·31	-0·06	-1·37	-0·97	-0·06	-1·62	-1·53
3·25	-1·21	-1·07	-1·27	-1·36	-0·70	-1·53	-1·14	-1·37	-0·20	-1·47	-1·03	-0·20	-1·65	-1·54
3·50	-1·29	-1·15	-1·34	-1·42	-0·81	-1·59	-1·22	-1·43	-0·33	-1·56	-1·09	-0·35	-1·69	-1·57
5·00	-1·75	-1·66	-1·79	-1·85	-1·41	-1·98	-1·70	-1·85	-1·06	-2·11	-1·61	-1·10	-2·06	-1·67

TYPE C

Fig. 12

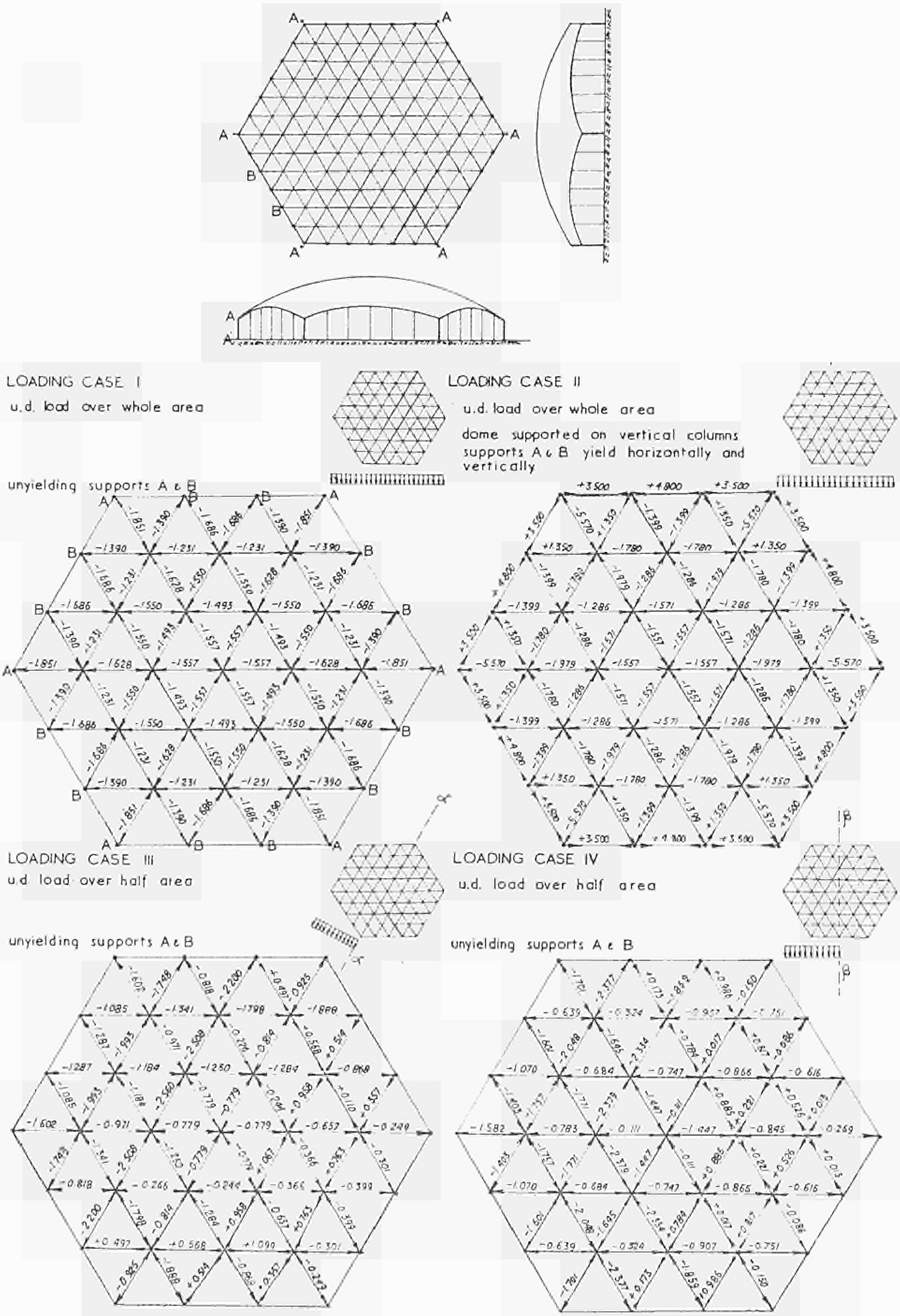

LOADING CASE I
u.d. load over whole area
unyielding supports A & B
LOADING CASE II
u.d. load over whole area
dome supported on vertical columns
supports A & B yield horizontally and vertically
LOADING CASE III
u.d. load over half area
unyielding supports A & B
LOADING CASE IV
u.d. load over half area
unyielding supports A & B

Fig. 13

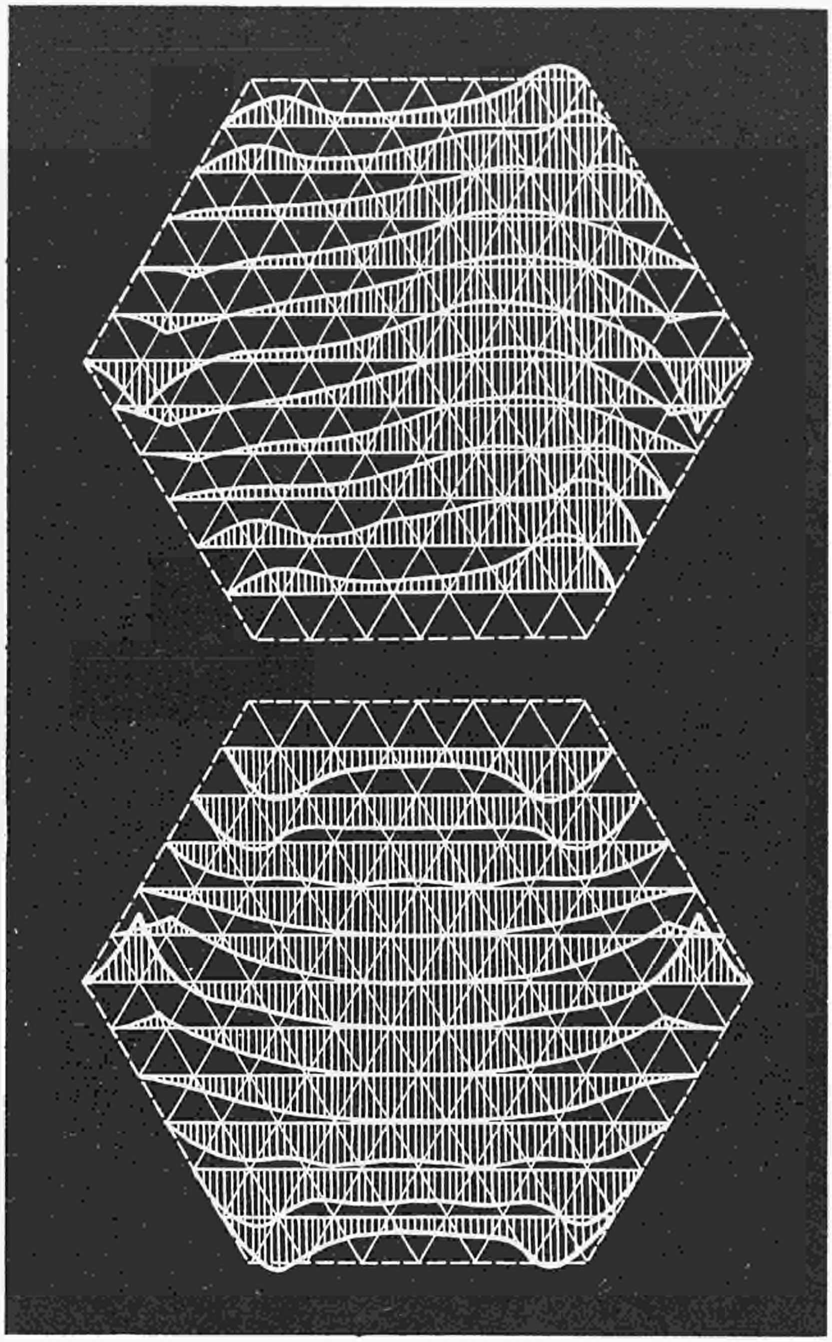


Fig. 14

Mr. F. West, another research student, is working on the stress distribution of a square tipped braced dome having a large opening at the centre. Various boundary conditions are being investigated and their influence upon the stress distribution determined.

Steel lamella domes are proving very popular in the U.S.A. A special interest in this form of construction has led to analytical and experimental investigations and the author of this note was fortunate in being associated with the analysis of the steel lamella dome built in 1964 in Houston, Texas, U.S.A. He acted as the consultant to the American firm of Roof Structures Inc., on the methods of analysis and the checking of the analysis of this huge dome. The structure has a clear span of 642ft. and is thus the biggest dome in the world (8).

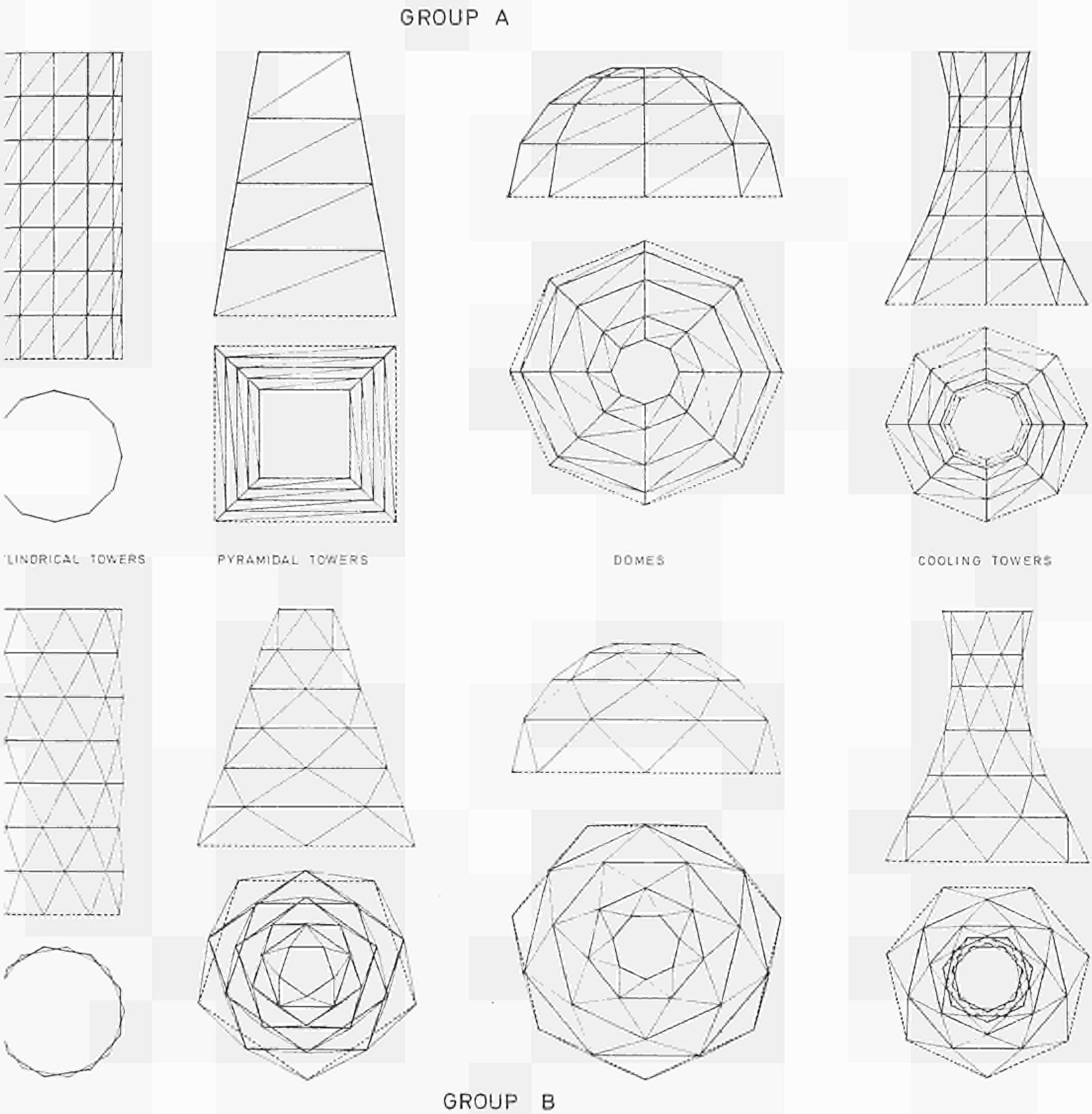


Fig. 15

The dome surface is divided into 12 sectors, each sector being divided again into 6 peripheral joints along the tension ring and 6 joints along the meridian ribs.

The net weight of this remarkable structure, including not only the lamella units but also the tension ring, is only 16.6 lb. per sq.ft., a very low figure for this record span.

The Space Structures Research Centre is also concerned with the analytical and experimental study of towers, in particular those used to support electric power transmission lines. Mr. A. Ramirez, a research student, is working in this field. There is also a research team consisting of six members of the academic staff, who are engaged in the preparation of a general programme for a transmission tower of a novel design, a research project sponsored by an industrial concern, Tubewrights Ltd.

In transmission towers it is necessary to consider vertical gravity loads, as well as horizontal ones caused by wind and broken conductors, which apart from dynamic effects induce bending and torsional moments in the tower as a whole.

In this investigation the influence of the geometrical shape, type of bracing, rigidity of connections and eccentricity of members meeting at the nodal points are taken into account, not only for the conventional type of towers, but also for new structural forms.

A general programme is now being prepared for the electronic computer for calculation of node co-ordinates suitable for structures consisting of members forming a regular pattern. Given the node co-ordinates, the lengths, direction cosines, projections, angles etc., are obtained for all members of the structure.

Special attention is given to particular problems such as the clearance between the members of a transmission tower and the electrical conductors, to comply with given specifications.

Two groups of structures are being investigated:

Group A — cyclically symmetrical structures formed by quadrilateral panels. The main types of structures of this group are shown in figure 15.

Group B — consists of cyclically symmetrical structuesr formed by triangular panels. These are obtained as a modification of Group A by rotating each horizontal ring through an angle of $\frac{\pi}{n}$ with respect to the ring below, where n = number of sides.

Pin-connected structures of this type with an even number of sides are theoretically of critical form and can resist loads only if their members are rigidly connected.

The computer programmes which have been prepared for such structures can be used for the analysis of pin-connected towers, but it is possible to modify the programme so as to take into account the influence of rigidity of the horizontal rings as well as the stiffness of the continuous ribs.

The programme is prepared in such a way that the contribution to the total strain energy of each stress resultant (axial forces, bending and torsional moments as well as shear) can be assessed separately.

It is also intended to study the shell analogy for such structures and the derivation of general mathematical expressions for a continuous medium representing the actual tower.

The problem of the design of a tower by means of the computer will be considered at a later stage.

A full-size prototype transmission tower designed in this way will be tested to destruction in July 1965.

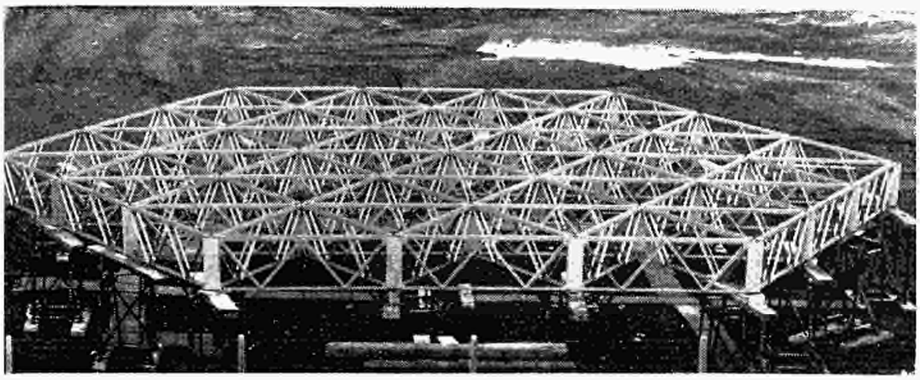
One should also mention briefly the work on pyramidal stressed-skin thin-sheet roof structures performed by two research students Messrs. R. Gilkie and D. Robak. One is already testing full-size structures consisting of prefabricated hexagonal pyramids, the other is developing a mathematical theory for roofs composed of square based pyramidal units.

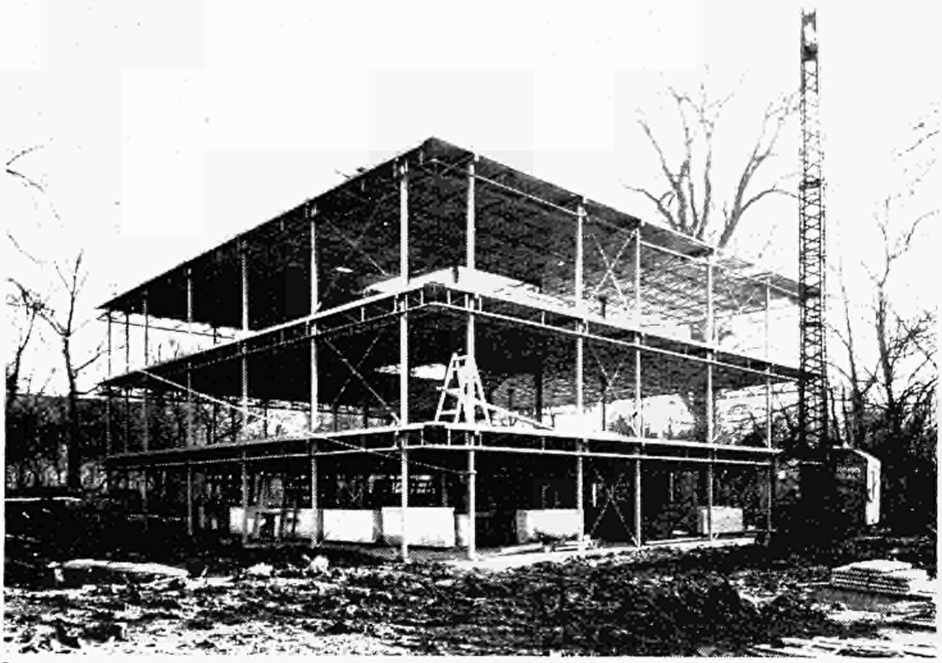
Another member of the staff, Mr. H.A. Buchholdt, is working on suspended network roof structures consisting of prestressed steel cables.

The work done on models of three-dimensional structures has shown clearly that even in this age of high speed electronic computers, there are still many cases where in space structures, especially those with complicated and unusual types of bracing, model analysis is probably the only practical answer.

However, one should issue a note of warning—the preparation of complex three-dimensional models for experimental investigations is very costly and time-consuming. Often it is impossible to reproduce in a small-scale model an exact replica of the real structure and as a rule the influence of rigidity of joints upon the stress distribution is over-emphasized in models.

Another point is that very little is known about the distribution of wind pressure upon the surfaces of space structures. The external shapes of such structures are often very complex and wind tunnel tests are essential if any precise determination of loads acting upon the framework is to be obtained.





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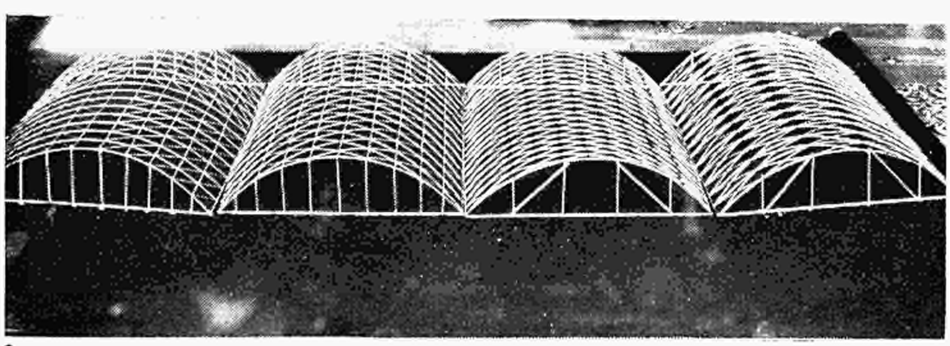

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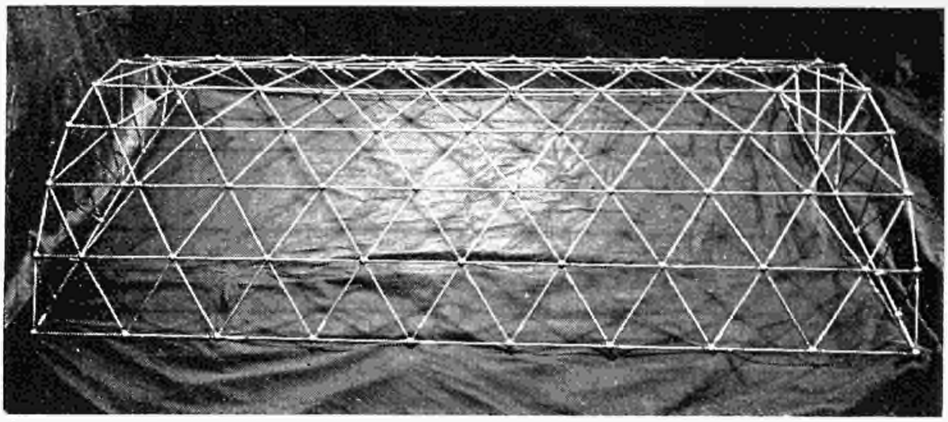

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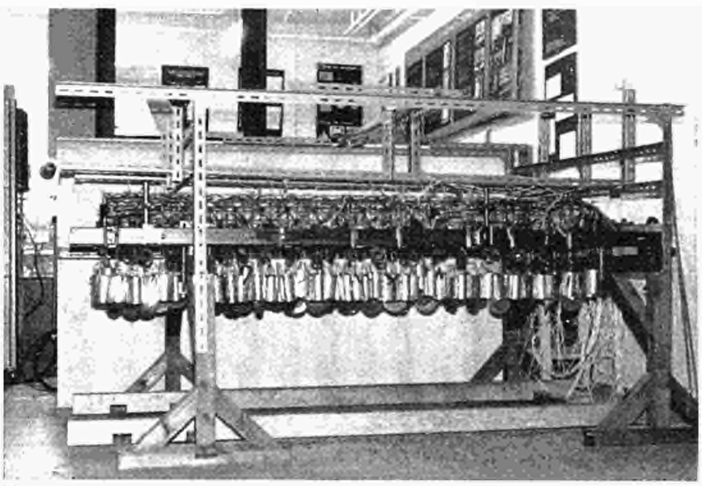

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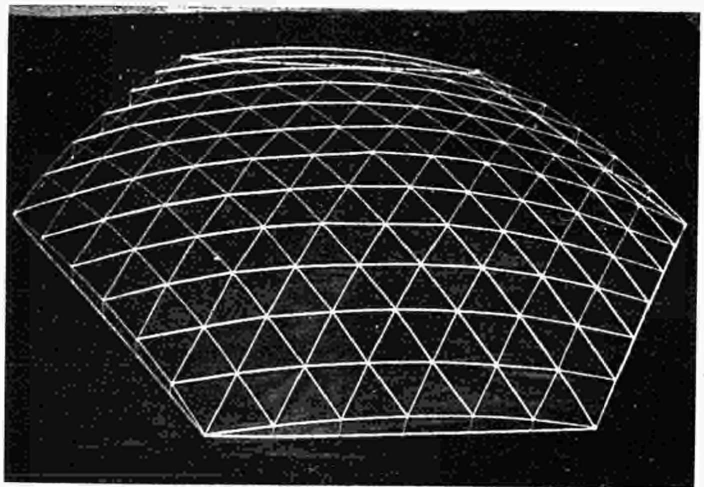
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Hermann BEER

Space-Frame Structures

(Original text: German)

I should like to ask Prof. Makowski a question. I entirely agree with him as to the economy of all curved or shell-type frame structures, but not altogether as regards the frames, that is, lattice girders or plane space-frame structures. The connections would have to be of extraordinary simplicity for the structure to work out cheaper than the traditional framework constructions. Now my question is whether it really is now possible to produce connectors cheaply enough to make these space-frame structures economically worth-while, even if they are not of the curved or shell type. My experience, from a number of comparative studies, is that constructing roofs in the form of single trusses, or splitting up into a shell, offers cost advantages, but that a space-frame structure arrangements as indicated by Prof. Makowski does not. How then is this enormous cost reduction achieved to make this mode of construction a more attractive proposition ?

Z. S. MAKOWSKI

The Cost of Space Structures

(Original text: English)

We always come to the question of cost. Economy is always of great importance in the design of any structure and space frames are no exception. It is true that in the past space structures were often more expensive than the conventional forms of structure. What are the reasons for this ? Professor Beer has quite rightly stressed this point. In the design of a space frame we face two problems :

firstly — the analysis and the determination of forces. If approximate methods of analysis are used, the designer is inclined to cover his own ignorance with a high factor of safety. This, of course, leads to an unnecessarily high consumption of material.

secondly — the problem of connecting various members at nodes. Because of the three-dimensional nature of space frames, the riveted connections used in the past have been very complicated.

During the last few years engineers and architects have concentrated on the development of various types of connectors for space structures. As a result of this, connectors are becoming simpler and less expensive. The least expensive method of connecting members in domes has been developed by Professor Lederer for his tubular three-way grid domes. His structures consist of curved steel tubes forming a triangulated space lattice. Where the three runs of tubing meet, they are held together by simple adjustable clamps without using any expensive fittings.

Space structures can be economical if designed in the proper way, and many of those that have been built within recent years have proved that for flat roofs of large span they can compete with conventional types of construction.

I, myself, have been concerned on a number of occasions with designs of space structures and it is interesting to see that many such structures have been built by people who are not in any way enthusiasts of space frames, but who selected this kind of construction because space frames proved to be some ten percent cheaper than conventional roofs.

In many cases the lower cost can be directly attributed to industrial methods of construction. Space frames can be built from small units. To produce them you do not need expensive equipment. It is a fact that even large braced domes have been built by small contractors, who have very low overheads. This is reflected in the final cost of the structure.

We did not mention the aesthetics of space frames, because this factor cannot be expressed in terms of money.

Roof trusses are ugly in appearance and this is one of the reasons why the architects are attracted by the visual beauty and pleasing regular pattern of members forming space frames.

Many modern steel space grids cover exhibition buildings and assembly halls. Because of the decorative appearance, the underneath of the structural framework is often left exposed. This falls in line with current architectural concepts which permit construction details to show.

Leo FINZI

Structural Problems Involved in the Development of Constructional Steelwork in Italy

(Original text: Italian)

The steel structures built in Italy during the last ten years reveal certain well-defined tendencies on the part of the designers, who have striven not only to compete with similar solutions in reinforced concrete but also to keep abreast of constructional and assembly methods which have changed considerably in Italy during recent years.

A number of points are worthy of mention :

(a) Generally speaking, with large spans and heavy loads there is an increasing tendency to build extremely compact monolithic structures with the components as far as possible, *i.e.* in many cases we have witnessed the disappearance of the traditional dense lattice type of structure and also the more recent solid portal frames in favour of very open-mesh lattice structures with members consisting of a single section or welded component. This tendency has been encouraged not only by the availability of self-supporting roof claddings covering increasingly larger spans and of an extended range of broad-flanged beams but also by the possibility of producing machine-welded composite members. This has been evident both in roofing large space-structures and in bridge and crane design.

(b) For structures with smaller static stresses, however, such as residential and industrial buildings with only a few storeys, the trend has been towards the "diffused structure", *i.e.* the prefabrication of room-sized panels complete with all fittings and with a framework of rolled or bent sections, as used by door- and window-frame manufacturers.

(c) Generally speaking, in many fields it has been found convenient to rely whenever possible on the torsional strength of the structure; extensive use has been made of this possibility in recently built bridges and cranes.

(d) The use of simplified connectors such as Nelson & Phillips rivets has favoured the spread of composite steel- and concrete structures.

(e) Increasing attention is being paid to methods of joining and connecting structural members. Since a high degree of simplicity and safety of site operations is required, ordinary or high-strength bolts are being used on an increasing scale.

These technological advances have made it necessary for experts and designers of structural steel to keep abreast of developments. Close attention is being paid nowadays to the post-critical behaviour of open- or closed-section light-gauge components, in order to ascertain safety factors correctly. In composite steel- and-concrete structures, on the other hand, interest centres on the effective behaviour of rectangular boxes of high torsional strength consisting of two parallel steel plates, an upper concrete deck and a lower latticework of wind-bracing.

Above all, however, it has now become imperative to provide a bridge between constructional practice and the limit theories of perfect elasticity and perfect plasticity.

Here the major and most urgent problem is that of beam-to-stanchion and beam-to-beam connections, not merely possessing known and guaranteed strength qualities but also of known and guaranteed elastic and anelastic deformation properties. Although in some buildings there is an increasing tendency to use a cross-braced structure, for example buildings which are less subject to horizontal forces can do without such bracing, and in all cases economic design demands the most rigid possible assembly of horizontal members and also of the latter with vertical members.

All the better, then, if ordinary shear-stressed bolts can be used rather than the friction-grip bolts now employed as a precaution, because the former are cheaper and make erection simpler.

The Italian Association of Constructional Steelwork Engineers, of which I have the honour to be President, is proposing to sponsor a series of theoretical and experimental studies in this connection.

Henri WAISBLAT

The Calculation of Structures in Stainless Steel Sheet

(Original text: French)

Builders may be interested in utilizing the high mechanical characteristics of 18-8 stainless steel sheet, but the classical methods of calculation for carbon steel are not directly applicable to the calculations for stainless steels. It serves to illustrate the point in the case of metal in the mild state and in the hardened state.

(1) For stainless steels in the mild state, the absence of a distinct yield stress on the load/extension diagrams has led to the adoption of a conventional E 0.2 [1] value, that is to say a value corresponding to an extension of 0·2% when the load is removed.

This value being thus defined, the calculations are carried out in the same way as for ordinary steel.

(2) By cold rolling, it is possible to make the mechanical characteristics vary considerably. The austenitic stainless steels display the interesting property of being less sensitive to brittleness due to hardening than certain other metals and alloys, their elongation at fracture being more pronounced, thus permitting them to be shaped.

Four rates of hardening have been arbitrarily defined: E_1 (1/4 hard), E_2 (1/2 hard), E_3 (3/4 hard), E_4 (4/4 hard), corresponding to the values of resistance to fracture and the yield stresses shown in the table.

To calculate members in hardened stainless steel, the difficulty resides in the fact that the material is anisotropic and that Hooke's law is only applicable in a different form: for a given metal, we have four values of Young's modulus according to the direction of stress (tension and compression longitudinal, tension and compression in a transverse direction), and the value of each modulus varies with the stress.

In addition, the anisotropy becomes more marked with the increase in hardening. Starting from the theoretical studies of Timoshenko[2], and on the basis of numerous practical tests, Watter and Lincoln[3] have been led to defining some practical values of the moduli of elasticity, taking into account the bi-directional stresses which develop in an element under compression or tension.

This research has provided us, for the purpose of calculation, with tools which are indispensable for putting this material

Yield stress values serving as a basis for the calculations of members in hardened Z 12 CN 18-8 quality steel.

Characteristics	Rate of hardening			
	1/4 hard	1/2 hard	3/4 hard	4/4 hard
	(Kg/mm²)			
Resistance to fracture	88	105	123	130
Yield stress for :				
— longitudinal tension	55	80	98	105
— longitudinal compression	47	60	67	70
— transverse tension	54	78	95	101
— transverse compression	56	84	105	113

to use. Certain study groups use a more empirical method, established as the result of tests, which gives results approximating to those of the scientific methods developed by these two authors.

A more noteworthy use of hardened stainless steel sheet is provided by the producers of railway rolling stock, who have been constructing coaches in hardened stainless steel sheet since 1937. These coaches are tubular structures shaped to section by cold forming and spot-welded. The advantages in reduction of weight and economy of maintenance are such that the use of these stainless steel coaches has spread throughout the French railway network.

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1 See Standard AFNOR A 03-101. Paragraph 6.

2 Timoshenko. Theory of Elastic Stability. McGraw Hill Book Co. Inc. New York 1936.

3 Watter & Lincoln. Strength of Stainless Steel Structural Members as a Function of Design. Pittsburg 1950.

Friedrich W. BORNSCHEUER

Increasing the Permissible Stresses for Structurally Loaded Fillet Welds in Steel St 37

(Original text: German)

In the Codes of Practice of most countries, the permissible stresses in structurally loaded fillet welds are, both for St 37 and for St 52, considerably reduced in relation to the values applicable to the base material; the same proportional reduction being applied to both steels. The regulations embodied in the DIN 4100 Code of Practice used in Germany provide an example of this. Here the permissible stresses in fillet welds are only 900 kg./cm.^{2} for St 37 and 1350 kg./cm.^{2} for St 52, whereas they are 1600 kg./cm.^{2} and 2400 kg./cm.^{2}, respectively, for tensile stress in the base material. The reduction factor is

$$\mu = \frac{6 \text{ permiss. (weld)}}{6 \text{ permiss.}} = \frac{900}{1600} = \frac{1350}{2400} = 0.56.$$

This considerable reduction is by no means justified in the case of St 37.

In general, electrodes of equal strength—with values in excess of 50 kg./mm.^{2}—are used for welding both St 37 and St 52. Tests on covered butt joints of St 37, which were carried out in eight countries within the framework of the activities of Committee X of the International Institute of Welding (I.I.W.), and supplementary tests in Germany with similar joints of St 52, showed that the failure stresses for both materials were approximately the same. Since fracture occurs almost exclusively in the fillet weld and since, with manual welding (which is predominantly used for structural connections in buildings), the base material mingles only at the edge zones with the weld deposit, this result—*i.e.*, equally high failure stresses for the two steels—should certainly be taken into account. Hence there is no reason for specifying the permissible stresses in the weld seams, as values dependent on the base material. As the average failure stresses found in the above-mentioned tests were very high—namely, over 6000 kg./cm.^{2} in edge fillet welds and hardly ever less than 4000 kg./cm.^{2} in side fillet welds—, the permissible stresses for fillet welds in structural steelwork could be substantially increased. Increases of about 50% are perfectly justified. Protests on the part of public authorities are hardly likely to arise, since structural connections in St 52 have, for a good many years, really been designed with a permissible stress that is 50% too high, namely, 1350 kg./cm.^{2}, although the load capacity of these connections is by no means higher than that of structures in St 37, for which hitherto only 900 kg./cm.^{2} has been allowed. Hence it can be said that real experience with increased permissible stresses is already available. For St 52 the stresses that have hitherto been allowed should remain unchanged. The economic gain in the case of St 37 would be considerable, and would have a favourable effect in the present keen competition with reinforced concrete. If the permissible stresses were increased by, for example, 50% the thickness of the seam could be reduced to 2/3, and the cross-sectional area halved. Apart from the substantial savings in material and in welding time, the reduced volume of the weld seams would also result in less shrinkage, thus reducing the troublesome subsequent rectification operations.

This problem was dealt with in greater detail in a paper presented at the major conference on welding engineering, at Wiesbaden, on 25 September, 1964. In that paper a design formula was proposed which is simple to use, but which takes more accurate account of the actual state of stress than do the highly simplified formulae hitherto developed. This new formula also enables economical solutions to be achieved. In order also to cater for complex fillet-weld joints, it is necessary to carry out supplementary tests, and preparations for some of these have already been made.

Entirely similar conditions occur in the flange welds of welded plate girders. In this case, too, available—but not yet complete—test results suggest that it will most likely be possible to increase the permissible stresses in St 37.

Hermann BEER

Actual Load Capacity of Orthotropic Plates

(Original text: German)

I would like to refer to Prof. Louis' comments concerning orthotropic plates. We are well conversant today with the elasticity theory of orthotropic plates and with how the individual elements are affected as well as how the plate behaves in the complete load-carrying system. However we know very little as to how big the actual maximum load capacity of the plate is in the complete system. We must assume that this plate has to fulfil three functions: the local transferring of the applied loads out to the main girders, which is the orthotropic plate action proper, the function as flange plate of the main girders for carrying the load out to the abutments and the function as the web plate in the so-called wind girder. The problem now arises as to the measures needed if we are to exploit the reserves of the orthotropic plate in the structure. We must consider the following points: firstly, the plate may not sustain any inadmissible deformation. This is very important especially with very thin deck surfaces which impart hardly any additional stiffness to the plate. We must therefore limit the deformation. Secondly, we must have regard to the local fatigue stresses which show up especially in the ribs; and thirdly, we must examine the problem of the stability of the plate when in compression as flange plate of the composite section. Finally, in our calculations we also must consider the aspect that local plasticizing can repeatedly occur and, I would point out, can produce in course of time elongation of the material. Consideration of all these factors points to the fact that we cannot fully utilize the high reserves shown in tests on individual elements. In spite of this, I am convinced that our permissible stresses are still too low and our safety factors too high—I am referring here especially to the German regulations. We can only produce a decisive change if we not only take elongation and deflection measurements but also estimate the behaviour of the structure on a long-term basis with consideration of all these points.

My second comment concerns Prof. Finzi's remarks on the use of hollow sections. We have carried out a thorough investigation into the economic limiting spans with orthotropic plates and with composite slabs and have established that the lower limits of spans, with the utilization of the reserves of the orthotropic plates at present allowed, are about 80 to 100 m. for these, and for the simple beam somewhat higher at 120 m. It is true that the composite slab construction, if we observe the most appropriate erection sequence, and especially if we use high-quality concrete, is extraordinarily economic.

I would now like briefly to express my opinion on the problems of hollow sections. We use these mainly with large girder spacings. The hollow section, and especially its lower flange plate can then only be fully utilized if the span exceeds a certain figure, because we are tied to a minimum thickness of plate. If we have a wide flange plate of minimum thickness, which in addition must still be stiffened, full utilization of the plate over the whole span is impossible. Prof. Finzi has referred to an intermediate member between the U-sections thus forming something between an open structure and a closed box. Bridges with lower wind-bracing are more economic for medium spans and reproduce the static action of the hollow box; they must carry additionally only the shear forces from the wind-bracing diagonals. It has been shown however, that the absorption of the shear flow is considerably more economically effected with diagonals than with a badly utilized plate.

Samuel CHAIKES

The Setting Up of a European or International Committee for Steel

(Original text: French)

I should like to make some comments on the proceedings of this Congress, as well as certain suggestions, but above all, I should like to emphasize the breadth and the interest of the papers presented, compliment the authors and thank the High Authority for arranging this Congress.

I have been agreeably impressed by the accent which has been placed at this Congress on the need for closer co-operation between the architect and the engineer, co-operation necessitated by the intensive development of modern techniques. Such a close co-operation is always evident in the better designed and more economical types of structure.

With regard to standardization, which is one of the themes of this Congress, I should like to express a guarded opinion. Although I am a partisan of the industrialization of building, I nevertheless believe that standardization should be used in moderation so that it does not become a two-edged weapon which is liable to kill all creative spirit.

I also wish to raise another point which I believe to be of great importance. I notice at this Steel Congress that there are representatives present from the Comité Européen du Béton (C.E.B.) and the Fédération Internationale de la Précontrainte (F.I.P.). There should also be a European or International Committee for Steel (C.E.A. or C.I.A.). I hope that such an organization will be set up. Derived from the national groups created in every country and wide open to engineers and architects, it could, like the others mentioned, co-ordinate research and disseminate the results obtained, and undertake the preparation of international rules and regulations.

Prestressed and reinforced concrete, which were originally separated into watertight compartments, to the detriment of both materials, eventually found common ground for co-operation as expressed in the setting up of a joint working party, F.I.P.-C.E.B., one of whose objects is to study and prepare a common standard for both these materials and their products. We hope that the Iron Curtain (if you will pardon the expression) separating steel and concrete will finally drop. The beginning of a *rapprochement* and even of a possible amalgamation between these two materials is already evident in composite construction, which has been discussed extensively at this Congress.

In practice, this new organization, open to all, would allow a more direct diffusion on an international scale of the techniques of steelwork and a useful method of working with other disciplines, thanks to contacts with their existing international organizations. The pursuit of the tasks for the advancement of construction techniques, where there is common ground, could be assumed, who knows, perhaps by a Joint Committee F.I.P.-C.E.B.-C.E.A. (or C.I.A.).

From the point of view of the Architect and the Engineer, this should permit a fairer choice of the right material for any particular case, eliminating the bias and prejudice which can result from the too exclusive practice of one particular discipline.

One last wish. The large number of Working Parties at this Congress has not allowed delegates to attend all the meetings which might have interested them. May I suggest that in the capitals and the university centres of the Community a cycle of conferences should be organized, under the aegis of the E.C.S.C., by the rapporteurs of the various working parties or by the authors, in order that the wealth of material produced during this Congress can reach all who might be interested by it.

Philippe DEMONSABLON

Model Tests and Design and Calculation of Structures

(Original text: French)

I should simply like to speak about three most interesting points which Professor Louis mentioned in his introductory report.

The place of model tests in the design of structures

As Monsieur Makowski said, the advent of electronic computers marks not only an evolution but a veritable revolution, because these instruments allow us to undertake by means of arithmetic calculation, the study of structures whose complexity is such that they can only be tackled by analogue methods. However, it does not seem that the existence of these new methods of calculation will completely supersede model tests, but rather give them a slightly different course. It seems that in the future model tests will be carried out on a large schale; this will, moreover, have the advantage of reducing the difficulties inherent in the introduction of scale effect, in particular those relating to internal stresses in welds or to research in the plastic range. Model tests will therefore be concerned with

(a) on the one hand, the full-scale study of fundamental problems (lateral instability, plasticity) or of fundamental elements, such as connections;

(b) on the other hand, the systematic examination of new structures, the object being to test the validity of mathematical models which would then be treated by automatic methods.

The introduction of probability methods in the design of structures

Professor Louis mentioned the divergence which can exist between the imposed loads assumed in design and those actually applied. I should like to mention, however, that certain standards, such as the French Standard for imposed loads on bridges, makes allowance for loads, the intensity of which decrease as function of the size of the loaded area. This specification is based implicitly on a probability concept, because the larger the area loaded, the less likely it is that it will be overloaded.

It should also be pointed out that probability techniques are being applied to the design of structures by Polish engineers, notably Professors Hueckel and Wierbicky. These two suggest that the factors of safety should be established as a function of three factors. The first is the scatter of the parameters defining the mechanical characteristics of the material; on the quality of construction of the structure and the tests carried out during construction and the size of the structure. It is obvious that the factor of safety can be less restrictive for a structure of average size or where collapse would have negligible consequences.

In France, these probability theories have been applied for eight years in tests on concrete by the Compagnie Nationale du Rhône.

Development of methods of calculation

Skew slabs

For several years, we have had at our disposal the remarkable methods evolved by Rusch after tests on models. I should like to mention that the Service Spécial des Autoroutes du Ministère des Travaux Publics de France (Special Motorway Section of the Ministry of Public Works) laid down two years ago an automatic method of calculation for skew slabs, based on bi-harmonic considerations and evolved by Professor Leray. This method can be applied to all bridges which are composed of a slab with a constant width perpendicular to the free edges, but in which the support conditions, such as the orientation of the lines of support, and the types of support may be of any kind whatsoever. Continuous slabs can also be treated by this method. The calculation programme provides the influence surfaces for the bending moments, shearing forces and deflections at all points, as well as for the support reactions.

Multiple cell bridges without bracing

My collaborators and I have recently developed a method of calculation for this type of bridge in France. It was evolved for the calculation of prestressed concrete, constructed in double curvature. In such structures, construction is virtually impossible unless bracings can be omtited. The distribution of non-uniform imposed loads is effected by the flexure of the connecting slab and the torsion of the box sections. The enhancement factors for the longitudinal bending moments are generally of the order of 5-8 per cent, from which it is reasonable to suppose that analogous construction could be used for steel structures where the employment of bracings would be a source of complication. This method, which is currently being converted into an automatic calculating programme, may be applied in particular to structures of variable inertia and to every case of loading, in particular discontinuous loading, for which analytical methods fail to give satisfactory solutions.

Jean BARTHÉLEMY

The Design of Large Structures from the Aesthetic Angle

(Original text: French)

I hope that the Working Party studying the design of structures in steel will forgive me if I touch briefly upon the problem of the aesthetic conception of structures, which, I feel is a fundamental one. I want to draw your attention to a fact which impresses me because it seems to be important for the future of large structures.

For centuries one man alone, the builder, was responsible for the design of structures.

During the last one hundred years, however, it has been possible to replace the old massive structures, the outlines of which were rigorously laid down by the necessity to introduce only compressive stresses into the constituent members, by steel space frames, the essential qualities of which are lightness and cleanness, thanks to their ability to resist considerable tensile forces.

This is a revolution whose consequences we still cannot fully measure even today.

Unfortunately, one of the consequences has been disastrous. The builder has not been able to retain his intrinsic unity, but has abandoned his poetry and intuition to the architect and his logic and science to the engineer.

What became of the protagonists after this spectacular separation of functions.

The architect, overwhelmed by the technical difficulties confronting him, lost his sense of direction and worst of all, as far as large steel structures were concerned, he, the champion of pleasing appearance, was condemned by his scientific ignorance to become a mere decorator. His art was relegated by events to the construction of balustrading and lamps and he made these subsidiary elements so monumental and so anachronistic that he often succeeded in wrecking the appearance of the whole undertaking.

But, what happened at the same time to the engineer? Happy to be in a new world where nothing appeared impossible, he kept his intuition intact and the schemes which he produced, although surprising for the age, were truly masterly. Just think of the Eiffel Tower and the Crystal Palace.

Then the enthusiasm faded away. He came up against great technical difficulties and he took fright, seeking refuge in methods of analysis and calculation which had then been evolved. He ended by conceiving stereotype structures easy to design. The beams and stringers in a bridge constitute a logical orthogonal frame greatly resistant to strain, but the scheme is highly statically indeterminate. For this reason, some of the members were overdesigned with respect to the others so that the structure would conform with simple hypotheses and, so could be easily designed. Such a procedure was, no doubt, evolved through the need for structural safety but not without producing other types of danger, such as atrophy of thought and destruction of constructional sense. But what is the position today?

The most recent scientific achievements in the field of design, such as orthotropic plates, focusing on the control of loads, boxgirders, mushroom slabs and stressed skins as well as in the field of experimental investigation, such as model tests, can give a new impetus and sense of direction in structural design. On the one hand, the proliferation of methods placed at the disposal of the engineer by science is increasing his vista enormously. On the other hand, all these innovations tend to the same objective, now perceptible; that of the greatest possible simplicity and the most formal purity.

The engineer is all of a sudden seizing the undeniable plastic beauty that the most modern gifts of science allow him to attain. He is zealous about them and full of enthusiasm.

By another train of reasoning the architect, having concluded that the external decoration of buildings is outmoded, has realized that he must return to the eternal architectural qualities: unity, harmony, balance, proportion, structural honesty, functional requirements and economy of materials.

By the same token, he has been ashamed of all these external trappings, whose nonsense he has grasped, and he has tried to rethink the fundamental problems of structures. Personally, I was not surprised to observe at the International Conference on Architecture in Salzburg, presided over by Professor Konrad Wachsmann, with what enthusiasm and faith many architects were setting about the task of designing new structures. That they are well prepared for this mission is a subject which I need not broach here.

This was the exciting development which I wanted briefly to discuss with you. Science, in guiding us towards more refined solutions, seems to invite us to take the further step, of letting our constructive imagination and our sense, of the aesthetic lead us on in new structural research and design.

Roger Alfred FOUGNIES

Latent Strength of Steel Bridges

(Original text: French)

If I have rightly understood Prof. Louis's remarks, it looks as if steel construction has potentialities which are not at present being sufficiently utilized, either because existing regulations do not permit any further exploitation, or because some aspects of the stresses involved are not yet fully known.

In support of this point, may I say a word concerning our own experience in this connection at the Bridges Division of the Public Works and Highways Department in Brussels, of which I am head.

For some years the Division has been receiving more and more applications for very heavy lorries to be allowed to cross bridges where the impact—on some components of the bridges at any rate—will be well above the load the structures were designed to carry. For instance, you get single consignments of 240 tons, which have to be driven separately across the bridge at reduced speed, with no other traffic passing, and there are already plans for loads of as much as 300 tons.

When we checked, we were nearly always able to give permission in the case of steel bridges, but we sometimes had to refuse with regard to certain reinforced-concrete structures, and very frequently with regard to the latest type of pre-stressed concrete bridge. The trucks then had to go round by a longer and more expensive route.

Now—and I am in a position to know what I am talking about—both the steel and the pre-stressed concrete structures had in each case been calculated in strict accordance with the standards and regulations in force. This bears out what Prof. Louis was saying, that the existing steel structures, both the older and the more recent ones, offer a further latent strength which some other types of construction apparently do not.

One may therefore well ask whether, to arrive at a proper comparison between different processes and materials, the time has not come for the regulations as to the calculation of steel bridges, and indeed steel structures generally to be amended in line with those now current for these other types of construction: that is, to obtain equivalent utilization of the respective materials' potentialities in both cases. Of course, this would mean that exceptionally heavy loads would have to be prohibited on steel bridges as well as on reinforced and pre-stressed concrete ones.

Another possibility, of course, would be to construct reinforced and pre-stressed concrete bridges of a size capable of carrying heavier loads than those now calculated for, and which have frequently been readily authorized to cross the majority of steel bridges.

Only if the one or the other were done, I feel, would it be possible to treat the different building processes and materials on the same footing, and to make an accurate assessment of the relative merits of steel and other structures.

Walter PELIKAN

Difficulties to be Overcome in order to Produce Fully Satisfactory Structures

(Original text: German)

The papers and the discussions which followed indicate that much the same pattern needs to be followed in the construction of steel bridges as in that of steel superstructures in order to make them a still more paying proposition and if these are to compete with other building materials.

In planning and carrying out our projects, we must make use of all new advances in scientific knowledge, but at the same time we must co-ordinate these with rational design suited to the material, and with the latest discoveries concerning the properties of the materials. Only in this way shall we produce fully satisfactory structures well able to compete with other building materials, and so help to ensure the increased use of steel in building.

There are however still difficulties to be overcome if we are to achieve this. I would like, in conclusion, to mention but two.

The great and long-established tradition of steel construction has frequently impeded progress because many engineers and designers have looked to the great models of the past and forgotten that rapid advances in building involve correcting and improving on the accepted models. In future the regulations governing the main characteristics of building in steel will need to be adjusted much more rapidly than heretofore to the latest discoveries of theory and practice. It is the duty of the technical colleges—whilst giving all due weight to tradition—to train students to become engineers able, not only to learn from tradition, but also themselves capable of contributing to the future by independent and critical thought whatever the particular field in which they work.

A second obstacle is that new knowledge is not arrived at without difficulty. In conjunction with the theoretical developments lies the need for considerable practical experiments—experiments the cost of which way well be considerable.

As we saw from the example of orthotropic plates—one of the most up-to-date forms of steel construction—we are nowadays able to determine their intersecting forces fairly accurately, but we do not yet know enough about their resistance to fracture, and how they can be rationally sized. We can only find out by means of a great many ultimate-load tests to ascertain their reactions to stresses produced by tension and bending and by compression and bending.

Such tests however are quite beyond the means of any one research centre. They require therefore to be carried out at a considerable number of centres working jointly, with supranational bodies responsible for their co-ordination and if possible contributing substantial funds. Only if this is done can one hope to achieve real progress both speedily and economically.

Findings

At the first session the Working Party discussed problems relating to the elastic theory and its application to steel structural frameworks. The rapporteur explained how, by making the fullest possible use of the latest knowledge regarding that theory, steelwork designers have effected savings in cost and weight in the construction of fairly wide-span roofs. This becomes possible by the use of tubular members, shell roofs, and roofs suspended from cables.

However, with regard to all such structures it must at the same time be ensured that the saving in weight is not accompanied by excessive differentiation of the structural sections, for this differentiation, and the attendant complication of the fabrication work involved, will increase the cost of fabrication. This in turn will not only reduce, but will indeed often wipe out the economics resulting from the saving in weight.

Consequently—and this point received particular emphasis in the discussion—the structural design of such buildings is especially important.

Some very good examples of real savings in overall cost are provided by conical shell roofs, which make it possible to economize both on materials and on fabrication shop cost. Here is an excellent opportunity to give structural steelwork the benefit of designs which had hitherto appeared to belong exclusively to concrete or prestressed concrete construction.

The situation is similar with regard to roofs shaped like domes or cylindrical shells. These present a pleasing architectural appearance, they enable the weight of steel to be substantially reduced, and—when designed for use with economical structural connections—allow of real savings in overall cost. This is confirmed by Italian and British examples cited by participants in the discussion.

At the second session, problems of steel bridge construction were, in the main, examined. It appeared that in this field, just as in building construction, the application of the latest scientific knowledge, combined with a design that allows of rational finishing, can produce satisfactory results from the viewpoint of economy.

In the lecture and in the subsequent discussion it was emphasized that, in virtue of a long tradition, many structural design methods that are obsolete in our present stage of knowledge are still widely used in practice and give rise to uneconomical solutions. Also, the official regulations, which are not always abreast of the latest knowledge, prevent more rapid progress towards a more profitable method of construction. Hence great importance must be attached to achieve speedier adaptation of the regulations to fresh knowledge than has hitherto been the case.

Load-bearing structures are largely designed in accordance with the elastic theory. As recent investigations have shown, however, structures designed in this way still have reserves of bearing capacity which can be brought into play before failure ensues. To develop the use of steel, it is most essential to utilize these reserves. The plastic theory that could enable them to be fully utilized has, however, not yet been developed to the point where it can yield generally valid directives for design. Therefore, in order to determine the actual strength structures, it is necessary to carry out tests involving leading to failure.

With orthotropic plates, which are well known and widely used in steel bridge construction, this failure load —it was asserted— is not merely a function of the material and the magnitude of the force, but also of the nature of that force. This means, however, that a very large number of tests will have to be carried out in order really to obtain a complete idea of the strength or the reserves of bearing capacity that remain as yet unutilized in these structures. It will be necessary to plan and carry out such tests under a scheme of co-operation between various organizations.

During the discussion another very important point was also considered, namely, the requirement that, in striving to construct as economically as possible, the aesthetic and artistic effect of the structure must not be lost sight of. In this connection it was stated that aesthetics and profitability are not necessarily incompatible, but that these two essential points of view will have to be co-ordinated by the examination of a larger number of variant designs. Examples from actual practice show that quite often in such attempts the most economical solution is also a perfectly aesthetic one.

Hence it is essential, more particularly in the case of important bridges and other comparable structures, that close co-operation be established between architects and engineers, and it rests with the universities and major training colleges to rouse the experts' interest in the matter.

In conclusion, the result of the two meetings can be summarized as follows :

Greater use of steel in the construction of brigdes and high structures is entirely within the range of possibilities, if such structures are designed on the basis of the most up-to-date scientific knowledge; but they must be so designed as to allow of rational finishing and erection operations. To clear a way to the practical application of the latest scientific knowledge, it is necessary that the official regulations for the design of steel structures be adapted to such knowledge as speedly as possible.

The application of the latest knowledge and the logical constructional utilization thereof lead to new structural forms (shell-type structures, latticework assemblies of prestressed cables), which can perfectly well be profitable if they are suitably designed and which can extend the scope of steel construction into a domain which had hitherto appeared to belong exclusively to concrete and prestressed concrete.

In the endeavour to construct as economically as possible, it is, in steelwork design, essential not to neglect its aesthetic and artistic aspect. In the long rung the construction of inexpensive but unsightly steel structures will be an obstacle to the extension of the use of steel as a structural material. Numerous practical examples show that profitability and good aesthetic appearance can go together. To achieve this optimum it is merely necessary to prepare a number of designs, which will lead to the most favourable solution.

The steady improvement in the profitability of steel structures constantly calls for fresh research—both in design theory and in the sphere of rational finishing operations—which, in order to guarantee their full efficiency, must not be carried out separately and independently in the various countries, but which, on the contrary, should if possible be directed by large supra-national organizations.

WORKING PARTY VII:

Building-Site Organization and Improvement in Productivity

Chairman :

France HÉBRANT

Rapporteurs :

Robert GARDELLINI

Prof. Dr.-Ing. Wolfgang TRIEBEL

Prof. Ing. Vittorio ZIGNOLI

The Working Party's subject was workshop rationalization and building-site organization to ensure maximum productivity; the main stress in its proceedings was laid on the need to have as much as possible done in the workshop stages in order to leave as little as possible to be done on the site.

The workshop must be turned as it were into an organized "factory", mechanized and if possible automated, so as to ensure that highly complex sets of structural components were turned out at lowest cost.

On-site operations must be reduced to pre-planned assembly and erection. This was bound to mean major changes in the pattern of the constructional steelwork sector.

Another point emphasized was the vital need for production planning right through the process from the architect's drawing-board to the post-erection finishing, based on time studies in respect of the principal operations involved. Research would therefore be required in this connection, and to ensure the application of the results attention would have to be paid to the training of technicians, whose functions were changing in line with the changes in the sector as such.

Robert GARDELLINI

Productivity of Building Activities as a Prerequisite for Raising Productivity

(Original text: French)

The concept of productivity, and its implications for the raising of the standard of living of producers and consumers, is of recent origin.

The idea of increasing output and its improvement is a long-established one the importance of which has always been apparent to employers (this term is used here in its widest sense to denote those exercising control over others and responsible for production of any kind). Archimedes, Vauban and many other famous names mark the advance made through the use of observation, experiment, practical work and invention.

But systematic and scientific work-study is a much more recent development. It dates from the time the Eiffel Tower was built, when steel came into regular use—a mere coincidence—no doubt as a result of the spread of industrialization, economic concentration and most of all, the introduction of repetitive operations.

The work of Taylor is generally regarded as marking the beginning of systematic research into the most economic organization of work.

It soon became clear that it was not just a question of reducing costs through the introduction of better working methods or of mechanization. The organization of the industrial set-up and of the management of business is also very essential. Indeed this is the primary requisite; something which builds up morale and internal conditions that are favourable to progress.

Fayol, nearly fifty years ago, made a first synthesis of this new aspect of organization.

Identification of all the factors contributing to a better use of natural resources and their most efficient utilization has given rise to the term "productivity". Since this is a comparatively recent expression, it is often misunderstood by many people.

Productivity means not only the organization of work, or the best way of investing capital, or the finding of the most profitable forms of organization or any other specialized technique. It comes as the result of applying all methods which enable one to produce more at a lower total cost.

After less than twenty years since people first became aware of this economic and social phenomenon in Europe, productivity—still in its infancy—can show, not only a credit balance, but a remarkable one. In Europe the standard of living has risen considerably in absolute terms and still more highly in relative values. The contrast between the economic stagnation of the inter-war years and the continuous progress of the last fifteen years is basically due to the sustained efforts, the systematic application of knowledge, methods and techniques that can properly be ascribed to this factor productivity.

The study of business concerns, of methods of work, and of productivity was at first naturally directed to the consumer industries. It is these categories which employ mass production methods and standardization.

Techniques are more established here than elsewhere, mechanization and automation are possible, work study is easier and is more likely to pay off.

Mass production has led to amalgamations and concentrations of industry, and is still doing so. The big concerns so formed have had at their disposal the facilities necessary for research much longer than have the smaller firms.

However, it is not just through increases in the consumer goods industries making these goods increasingly plentiful and cheaper that the purchasing power of the community increases. Efficiency in agriculture and in the distributive trades is just as important. A change, though not such a spectacular one, is also in progress here.

Improved productivity in the capital goods industries is equally important and will be increasingly so as time goes on.

Indeed, in proportion as the value added directly from the processing of materials diminishes so the relative cost of paying for the capital equipment used in the process increases.

The result is that the search for methods of reducing the total cost of capital goods is becoming more and more the concern of the day.

Buildings, machinery and equipment, all of which affect production costs, need to be scrutinized and systematically investigated by us.

Productivity in steel construction

In most European countries, public authorities and national associations. working in different ways on parallel lines, and sharing their findings, have encouraged, helped and financed this work for productivity, whether conducted on an industry or a regional basis.

In France, and in the industry that concerns us, namely, building in general and structural steelwork in particular, we have derived great benefit from the assistance of official bodies, thanks to a close permanent collaboration between the industries and the Commissariat Général du Plan d'Equipement et de la Productivité (General Commissariat for Re-equipment and Productivity).

This body started the move towards greater productivity when it organized in 1947 fact-finding missions to the U.S.A. These study tours led to the formation of associations for the study of productivity by industries or by voluntary groups of firms.

The National Building Federation of France set up "APPROBA".

A group of firms, on the initiative of some industrialists who had visited the U.S.A., formed in 1953 the CEPCM or Centre d'Etudes de la Productivité dans la Construction Métallique (Research Association for Productivity in Steel Construction).

The Centre de Productivité de la Chaudronnerie (Association for Productivity in Steel-plate construction) was formed in similar circumstances.

After approving the working programmes submitted by these voluntary associations, the State helped to finance their studies over several years. It still assists various investigations of general interest.

It would not perhaps be out of place to recall that, in the case of at least one of these associations the decision to set up a permanent working party for a prolonged period was influenced in part at least by one of the observations made in the U.S.A. The factories visited there were not markedly different from those of European factories, the machines seemed to be virtually identical, the performance of the workers was not in any way outstanding, and the materials used cost about the same as ours. Yet the selling prices were similar to European prices despite the much higher wage costs.

In face of this seemingly inexplicable paradox, our industrialists came to the conclusion that only an exhaustive and lengthy research would enable them to find ways of achieving similar results.

Their hopes have been borne out by results.

Their initiatives have encouraged the formation of other working parties and moreover, their activities and scope of these have expanded.

We would like to mention the very valuable achievements of the Belgian structural steelwork and sheet-metal working firms. The inter-industry organization, Fabrimetal, after investigating the methods and results of the French productivity associations for steel construction and sheet-metal work, was instrumental in forming a similar group, which the official body for promoting productivity, known as OBAP, encouraged and assisted.

While on the subject of this expansion, we ought to mention, in connection with French legislation on industrial technique research centres, the initiative taken by the structural steelwork industry. Within the Centre Technique de la Construction Métallique (CTICM), that is, the Technical Centre for the Constructional Steelwork Industry, a research service that deals with work study and technical assistance for all firms in this industry enables these to benefit from investigations and methods originally intended for founder members only.

I must apologize for quoting only a few of the best known cases. Similar developments have taken place in other countries represented here, and we shall certainly hear of some very instructive examples in the course of this meeting.

Without wishing to anticipate the reports that are to follow these introductory remarks, it would seem significant that the results obtained are the more significant and lasting when action is taken simultaneously on three fronts :

— the basic and advanced training of the workers,
— the general management of the firms,
— work organization and job techniques.

Productivity is in the first place an attitude of mind, and not only at the level of the leaders of industry.

Adaptability to change (improvements in techniques and methods) is perhaps the most difficult attribute to acquire and to impart to others. That is why it is always necessary to start by creating a climate of productivity at executive, managerial, and at shop floor level.

All these productivity associations have realized this need. The programmes of the bodies already mentioned have been exceptionally well geared to requirements. One has only to note the number of seminars and study group meetings and the programmes of these sessions to appreciate this and to be encouraged by their example.

The management of business is the necessary preliminary to more intensive technical studies, because a certain level of "industrial civilization" is essential to the establishment and proper functioning of the machinery for raising productivity.

It is no mere chance that the first publications issued by the Technical Centre for the Constructional Steelwork Industry should bear such titles as :

"Recherches sur la dimension optimale et l'adaptation au marché des entreprises de construction métallique" (Investigating the optimum size and marketing potential of structural steelwork firms);

"Structures et direction des entreprises de construction métallique" (Organization and management of structural steelwork firms);

"La gestion des stocks en construction métallique" (Stockkeeping in the structural steelwork industry).

Among the sixty publications which the CEPCM or Research Association for Productivity in Steel Construction has issued in ten years, more than ten have been devoted to management. The same applies to the other curves of production.

Among these studies of general interest we schould mention those dealing with cost accouuting—the tool of the executive—bookkeeping, cost analysis, production costing, budgetary control, statistics, charts and curves of production.

This indicates a deep, realization of the fact that without the ability to look ahead, without the means of assessing and controlling production, and without a systematic investigation of the disparities between achievements and forecasts, it is impossible to enjoy to the full the benefits of technical progress and of organization and method.

Finally, the third front element of productivity is obviously all the studies that bear on the organization of work and industrial technology.

The study of plant layout, operations, work shifts and team work are all other means of reducing costs and of working to schedule, of improving quality and of reducing the amount of physical exertion for the workers —four ever-present and inseparable objectives.

These studies have often been so thorough as to be instrumental in bringing on to the market new, improved and fully automated machinetools; we shall certainly hear of such cases.

These are leading, especially in the field of constructional work, to a rethinking of the design of structures, either because of developments in technology or because the employment of these developments is brought within the reach of a greater number of work-shops (welded designs, extended use of plates and sheet) or because increased output and the interest of customers impose such a solution (standardization of construction, industrialized building).

The study and practice of productivity is therefore many sided. It goes far beyond what is often caricatured as mere "simplification of work", whose value nevertheless remains, but which forms only a very small part of what has been accomplished over the last fifteen years.

The problem of construction sites

The major problem for all those who look for improvements in productivity in building work obviously concerns site work.

This is the work place for most of the building industry. It is the activity which can be seen by clients and by users generally.

All these are surprised to see on these sites more and more powerful plants, and yet an army of workmen, activities that seem to be poorly co-ordinared, in total contrast to what anyone can observe in field workshops and factories.

This field is therefore a very good choice for studying productivity and for finding ways of increasing it, but it is at the same time very difficult territory.

A workshop has a fixed location, regular organized working shifts, and repetitive jobs, at least at the level of the elementary productive operations, even though the end product, made to order, is always different.

A building site on the other hand is exposed to the vagaries of the weather; is never exactly the same as any other, and the work is mostly manual and not repetitive.

The man in the factory works at one continuous job, or, as part of a stable permanent team. Management is close at hand and the work is carefully planned.

On the building site the teams are numerous and vary in their composition, men move from gang to gang and supervision is limited because it has to cover a wider area. Even the idea of planned work operations is less in evidence and less rigorous.

Supervision is difficult, and the drafting of systematic rules and methods of working calls for very lengthy study, the financing of which does not offer any immediate return. Last but not least, work on building sites tends to leave more responsibility and latitude to the man on the spot. Without exaggerating unduly one might say that each does what he can, how he can, with much improvisation and makeshift. Site organization still relies too much on the individual resourcefulness of chargehands rather than on the organizing ability of business executives.

Since this state of affairs has existed for a long time and men on building sites have had years of experience of this absence or inadequacy of method, it will clearly be hard to wean them from it.

All these factors account for the slower rise in output on building sites as compared with the factory floor over recent years. In steel construction this has reached the point where, unless these difficulties are soon overcome, more time will be spent in assembling and erecting steel frames and other structures than in fabricating them and doing all the preparatory work in the shops.

In the building industry generally, experts have been working on this problem for more than ten years. In structural steelwork until recently we have had only isolated examples of productivity studies. The French Productivity Association for Steelwork Construction, for example, thought it better to give its attention first to productivity in the workshops and started only four years ago on a more general and comprehensive study of site conditions.

Arrangements on building sites are certainly not as bad as they were before the war: mechanical transport of ever-increasing capacity is making it possible to use bulkier prefabricated units. More powerful cranes can handle heavier and heavier loads. Both help to raise productivity. Nevertheless, the detailed study of site work still seems to be in the experimental stage. First steps have been taken and some spectacular results have certainly been obtained—we shall have confirmation of this by the end of this meeting—but these successes have not yet managed to raise the average level of productivity on building sites.

The merit of this Congress will be that the exchange of experiences, and the publication of the evidence that is collected, will enable everyone to have the advantage of the best ideas and methods.

Experience in France shows the need at the moment for a very detailed analysis of site operations, a matter still barely understood. Dr. Duval will show us the value of action shots, taken with a ciné camera, for this work.

French experience also goes to prove that methods of work planning can be adapted to work on site. They assist and standardize the work of engineers, foremen and chargehands.

Finally, it makes the point that no lasting advantage will accrue without the continuous training of workmen and particularly the youngsters who tomorrow will exercise authority on the sites.

It is to be hoped that the reports of these investigations, still restricted to members of the Research Association for Productivity in Steel Construction, will be published as soon as possible for the benefit of the whole industry.

These include :

— Analyses de chantiers (Analyses of site operations);
— Manuel de préparation des travaux de chantiers (Handbook on planning site operations);
— Mémento pratique du chef de chantier (Practical notes for site managers).

Conclusions

The stepping-up of productivity in the French structural steelwork industry in general is a continuing process. A small percentage increase can be noted each year from the annual production and employment statistics. Fewer and fewer hours of labour are required to get a ton of steel into its final position.

This progress is the more marked as improvements in the use of steel and in methods of utilization permit a progressive reduction in the tonnage necessary for any particular structure.

The Eiffel tower, if it had to be built today, would be incomparably lighter.

Yet, what is more pertinent and of greater value for the future, is that the firms who have made a thorough study of these problems obtain much more remarkable results than the others.

It is not unusual for a firm in a particular line of construction to produce today, after five years of continuous application, twice as much with the same labour force.

We shall certainly be supplied with more detailed examples along with relevant explanation as to how such results have been obtained and one can only regret that these examples will still too often relate only to isolated instances and not to universal practice.

It must be said that, in our free countries, where the economy is based on mutual consent, research is almost invariably the initiative of individuals or, at the most of a group of volunteers animated by the same desire for progress and service. The idea eventually spreads and snowballs. Results become more and more important and more generalized, and it is at this point that they have their repercussions on the industry thus assisting its growth, and on the community, thus raising its standard of living.

The time has come when it is necessary to compare methods and results, but is this not precisely the object of our Congress?

Could it not also be the starting point for joint research, at Community or, indeed, international level, into the problems posed by steel construction?

Admittedly it is a difficult and lengthy task but is this an excuse for not embarking on it with increased determination and vigour?

The sharing of experience, the framing of a joint research programme, and the setting-up of a plan for financing the proposed studies are all calculated to yield concrete and speedy answers to the problems facing us and it is to these ends that we must apply ourselves.

Wolfgang TRIEBEL

The Reorganization of Building Activities as a Prerequisite of Raising Productivity

(Original text: German)

The performance of the producing side of the building industry does not depend on one party alone, as it does in an industry with a fixed operational base. In building, and more especially in constructional work and housing, a number of individuals, organizations and enterprises have a share in the operations. Each of them carries out only a part of the work: promoter, client, government or local government officials, architect, engineer, supervisor and contractors for the various specialized jobs involved. For structural steelwork we have to add to the list of enterprises engaged on the project the fabricators of the steelwork and the firm that erects it, sometimes two quite separate firms.

If productivity in the building industry is to be raised by any appreciable extent, it is not enough for just a few of all those who take part to conduct their business on more rational lines than in the past.

By reason of the manyfold sub-division of the productive function and the many different types of building, no single one of these activities and no single building component in itself forms a critical part of the whole. Even if individual tasks were organized on much more productive lines, the effect overall would be negligible. What is really necessary is that everyone concerned should apply his efforts to rationalizing the work as a whole and to increasing productivity, and that all operations should be carried out more efficiently.

These many operations can be classified under several headings the most important of which are :
- the designing of the buildings with an eye to economy,
- the efficient preparation of the work,
- the adoption of improved working methods,
- the rationalized organization of building sites.

With the aid of one or two practical examples under each of these headings I shall now explain how individual operations can be more rationally carried out and show what measure of success can be expected in each case.

The more productive performance of individual tasks

Designing the buildings with an eye to economy

The cost of building two flats of similar size and class of construction can vary within wide limits according to the type of house in which they are situated. The type of house which partly decides the building costs, may be determined by the number of floors, the pitch of the roof, the number of flats per floor and by whether the house is terraced or detached. Our first example shows how some of these factors determining the type of the house affect the costs of building flats that are otherwise similar.

If a comparison is made between the cost of building

(a) one flat on each floor of a detatched two-storey house with a steep roof and unused roof space, and

(b) that of a similar flat in a three-storey terrace block with two flats per floor and a low-pitched roof.

It is found that flat (b) is constructed at 75% the price of flat (a).

Thus as a result merely of minor alterations in these four factors which determine the type of house—and they could occur anywhere— building costs were changed by as much as 25%.

Building costs may be expected to depend on the type of house concerned, irrespective of whether the houses are built by traditional methods, or with prefabricated components, or as steel-framed buildings.

Differences in the building schemes, in individual requirements and in town planning conditions necessitate different building plans. Yet, even in differently designed buildings many parts, such as stairs, pipework, balconies, parapets, etc. are identical. If, for each of these items, the ideal, universally applicable design were developed, it would be possible to plan the buildings more accurately and to rationalize the manufacture of these items on mass-production lines.

The sanitary installation of a large residential block containing flats of varying layout, with individually designed plumbing arrangements, was estimated to cost DM300,000 (approx. £27,000). Some of these installations had been efficiently designed; others less so; but generally speaking they varied considerably in their layout. At the same time, the best designs for these installations were worked out with a view to standardization.

When these many different installations had been replaced by standardized, more efficient systems, the cost was reduced to DM200,000 (approx. £18,000).

The use of standardized, highly efficient service installations, mass-produced for houses of quite different types had thus resulted in a saving of over 30% on the costs.

Efficient preparation of the work

Thorough preparation of the work as regards both the drawings and all technical and operational details is essential, if subsequent operations are to proceed smoothly, without loss of time, hindrance or error.

Such preparation may involve extra work both, in the design and erection stages, but it helps to obviate the even higher unproductive expenditure that would otherwise accrue. Of the various tasks required for the careful preparation of building operations, I will give only one example, which illustrates the importance of properly prepared drawings. These comprise general arrangement drawings, detail drawings, assembly and erection drawings and details of service installations, which must be complete before building work commences. They must correspond in every detail and must not be subsequently revised. For steel-framed buildings they must be available in their final form before the steelwork is fabricated. Alas, this obvious requirement is not complied with everywhere.

Thus, as in the case of one major building project, inadequate preparation and premature commencement of work may almost be regarded as typical of many other cases. Merely as the result of subsequent changes, delayed preparation of detail drawings and procrastination in making decisions, the work involved in cutting away and making decisions, the work involved in cutting away and making good amounted to 6% of all the man-hours worked on the contract.

This extra work was necessitated in particular by :

(1) delayed decisions regarding the method of heating, so that chimneys had to be built as an afterthought;

(2) alterations in kitchen layout entailing changes in door positions;

(3) subsequent provision of attic rooms, involving opening up window space in gable walls;

(4) setting out of reinforced concrete staircases before completion of detail drawings, entailing subsequent alterations;

(5) changes of mind regarding floor finishes of landings, involving alterations to depth of screeding, etc.

To this avoidable expense of cutting away must be added the equally avoidable one of making good the parts cut away. All in all, delays in planning caused a waste of approx. 10% of the man-hours spent on the contract.

It is obvious from this that careful, realistic and timely planning of the building in every detail before work commences, together with refusal to admit subsequent alterations in the preparatory stages, will obviate the need for much unproductive work. If a carefully prepared set of drawings is a prerequisite for solid brickwork construction and pays for itself, it is absolutely indispensable for steel-framed buildings. The drawbacks arising from delays in providing drawings would in such cases be hardly tolerable.

Improved working methods

Of the many efficient new working methods that have been evolved, tried out and introduced over the years, we shall here discuss only one, which concerns the installation of pipework. It is of special interest because it belongs to one of the sectors of prefabrication of building components. The service installations partially prefabricated by this method are being used in buildings of load-bearing brickwork as well as in steel-framed buildings and in those made of large prefabricated parts.

By the former craft methods every piece of piping for gas, water and sanitary services was measured, cut to length and fitted individually on the building. This required a great deal of manual work which had to be done under unfavourable conditions.

By the modern method, the plumber takes measurements once only on the building for a number of similar rooms. He then cuts all pipes to these lengths in the workshops in sets, assembles them into complete pipe runs and fits them in the building. The work of assembling the pipework is simplified by the use of patterns, models, templates and jigs in the workshop.

In large experimental blocks, 0.46 hr per ft. run (1.5 h/m.) was required for installing gas pipes by the old method, but with the new method only 0.21 hr (0.7 h/m.). The corresponding times for water mains and pipes were 0.46 hr. (1.5 h/m.) and 0.18 hr (0.6 h/m.) respectively. Waste pipes needed 0.61 hr per ft. run (2.0 h/m.) by the old method; with the new one they were installed at a rate of between 0.21 and 0.30 hr (0.7 - 1.0 h/m.).

Thus, the effect of the new method was to eliminate roughly 50% of the normal working time.

Many other building components and fittings can be prefabricated to standard measurements, so that the whole internal work for the building is simplified, expedited and, in many cases, reduced in cost.

Rationalized organization of the building site

The arrangements that make for more efficient work on the building site itself consist of an appropriately planned sequence of the various operations, the employment of the most suitable machinery, a rational layout of the site, repetitive execution of identical operations by the same teams of workmen in long production runs, and so on.

All these measures serve to eliminate unproductive work and to raise productivity. The same performance is attained with less expenditure of labour. The success achieved by rationalized site organization and the adoption of a number of such measures is illustrated by the following example :

In the construction of a rather large housing scheme a number of identical terraces had to be built. They were planned by the same architect. Conditions were comparable throughout, the only difference being that the individual terraces were built by different contractors. One contractor had laid out his site efficiently. All he used was a crane. This was so located that it could of itself handle and move anything not involving much sideway movement. It could straddle the line of supply for building materials and unload all lorries. It was also used for assembling prefabricated components on site. The concrete mixing plant was so efficiently arranged that it could be operated by direct, short runs. It was so located that the crane, without travelling under load, could take the concrete from the mixer and deposit it anywhere. This firm also carried out many operations to a timed sequence.

Another firm of contractors, building an identical terrace of houses, employed in addition to the crane a concrete pump for concrete cellars and ceilings. For the same amount of work this firm used two expensive items of equipment whereas the first firm managed with the crane alone. Arrangements on the site were such that the crane could not itself handle all the transport of materials, some of which had to be done by hand. Nor could the crane unload all lorries. The concrete mixing plant was so arranged that the crane had to travel with each and every load when supplying concrete to the distant sides of the building. Working to a timed sequence was not adopted.

The first contractor needed 18,000 man-hours for building the carcass of his terrace of houses. The other did the same work with 22,000 man-hours. The 20% difference was due simply to several minor advantages in the way the site was laid out, in the use of equipment and in the timed sequence of operations.

The need for co-ordination

If, however, all the knowledge we have of rationalized building and increased productivity is to have any effect, all concerned must work intelligently together. The various activities which enter into the planning, preparation and erection of buildings are interdependent. Promoter and client must give the architect, and architect and surveyor must in turn give the contractor, opportunities to use their own special knowledge and experience. Local authorities must co-operate by constructing roads and providing basic services. The operations of the various sub-contractors working on the same project must be similary co-ordinated. When building with prefabricated components or in steel construction this is even more important than in the case of other methods.

The contractor carries out a given project with specified materials on a site the accessibility and facilities of which are mostly outside his control. True, he may control the economic side of the contract by his site organization, by the effective use of equipment, by appropriate composition of his teams of operatives and by purchasing materials at advantageous rates. The effect of all this on the rationalization of the scheme as a whole, however, is limited.

The possibilities for raising productivity, which we have just illustrated by a few examples, become effective only when the other parties concerned carry out their share of the work in such a manner as to enable those following—ultimately the contractors who undertake the work—to achieve higher productivity in their particular operations.

Co-ordination of the various activities in building is the prerequisite for effective increase in productivity.

For steel-framed buildings this entails the preparation of simple and clear drawings. These must be complete and final in every respect with all details and layouts of services before the fabrication of the steelwork commences. The complex internal items should be prefabricated to exact sizes in a limited range of shapes to give long production runs. Roads and external services must be ready before building work commences. A little time is required to produce a well-planned layout of the building site. The programme of work before erection, the erection process itself and the following main building, fitting-out and finishing operations must, in consultation with all contractors, be so governed by a programming and progress schedule in which all trades follow in their turn, without pressing and without hindering.

Practical applications and results

The satisfactory results achievable by co-ordinating building activities can only be demonstrated by practical examples. Such typical examples are also necessary if the advantages of co-ordination are to be made clear to the general public; if they are to serve the purpose of raising productivity throughout the building industry.

In the Federal Republic of Germany this task is performed by the "Demonstrativbauten" (demonstration buildings). In some of the *Länder* of West Germany the same purpose is also served by a large number of small projects, known as "Beispielbauten" (approved type buildings).

The "Demonstrativbauten" form compact housing estates, well laid out from the town-planning point of view, each comprising some 600 to 1,000 dwellings and the requisite community and service buildings. In the construction of these buildings, the principles already mentioned, that is to say, economic planning of the units, rationalized building methods and efficient organization of operations have to be strictly followed.

Everyone concerned (developer, employer, architect, engineer, supervisor, local authority, contractors and others) has to work together in such a manner that each gives the other scope for building on rational lines.

The work must be carried out non-stop within three years so that during this period, one group of dwellings after another is commenced and, later on, one after the other is completed.

Already during the building of one of the first of these estates, costs were seen to be lower than elsewhere, thanks to the efficient arrangements made before work was actually started. The operations were actually most carefully planned. All building plans and detail drawings were ready before work commenced and were not subsequently altered. Roads and basic services were completed before the buildings were started. The buildings themselves were of simple design. A large proportion of the fitments, such as stairs, services, balconies, were identical for the buildings. Although the site was occupied by houses of various types, one-family houses, four-storey blocks without lifts and eight-storey blocks with lifts, each contracting firm was given the order for one type of building only. Contractors were thus able to hand over the same jobs to the same teams of workmen for steady repetition work, one operation following the other in a timed sequence.

The whole project was carried out in six sections, each taking six months, and was completed in three years of uninterrupted, continuous working. As soon as the first section was finished, the contractors were able to build the five remaining sections according to the same plans on the same site, with the same teams of workmen and the same equipment.

Although building costs were rising everywhere while the work was in progress, the various sections of this demonstration site were completed at the old costs. At the end of the three years of uninterrupted building activity, these costs stood between 17% and 21% below those prevailing elsewhere. Moreover, not only were the costs lower, but fewer man-hours were needed, these working out some 30% less for the last sections than for the first.

Thus, thanks to know-how, organized co-operation by all, and continuous working, it was possible to produce buildings of high quality at a substantial saving in labour and costs.

Co-ordination of the various activities involved in building has thus proved to be the prerequisite for turning all the know-how as regards more rational and productive building methods to profitable account. It has already produced some notable results. Such results may be achieved wherever the right approach is adopted, irrespective of the building systems, or of the materials and practices employed.

This co-ordination is valuable for the more rational development of orthodox building methods. It is necessary for all prefabrication-based building, and for building in steel it is indispensable.

Vittorio ZIGNOLI

Technique, Economics and Organization of the Erection of Steelwork Constructions

(Original text: Italian)

Introduction

The economic study of the increase, with time, of the costs of various products which manual labour puts at the disposal of the community shows very clear differences for certain well defined categories.

The cost of living has a definitely growing tendency to rise. This is due partly to a gradual improvement of the standard of living which leads to increased consumption of various foods causing an increase in the cost of certain foods which are difficult to produce industrially. For instance, the consumption of meat which —owing to the notable increase in wages—is becoming ever more widespread throughout the entire social class structure.

Artisan production follows, and sometimes exceeds, the gradient of the average cost of living.

The cost of raw materials, which owing to increase in wages shows a sharper rise than of the cost of living, does not generally have gradients flatter than in the case of foodstuffs. In some way, the savings obtained by technical progress, (mechanization) have been partly offset by increase in the cost of labour and by the need to use ever more distant sources owing to the local ones becoming exhausted. On the other hand, mass produced goods show a clear and far less tendency to steepness as is clearly proved, for example, by the pattern, through time, of the cost of utility motor cars. (Fig. 1).

In any case, with regard to the cost of building materials, we note a considerable decrease in the prices of those products for which mass production is possible and has been developed. (Table 1).

The building industry shows two quite different tendencies : Large constructions for which it is possible to organise highly mechanized building yards, are similar as regards costs, to mass-produced goods. As an example, we mention the cost per cubic metre of concrete in the large dams of the hydro-electric installations. On the other hand, industrial and residential buildings follow a course which is close, especially in the case of houses for renting, to that of artisan production (Fig. 2).

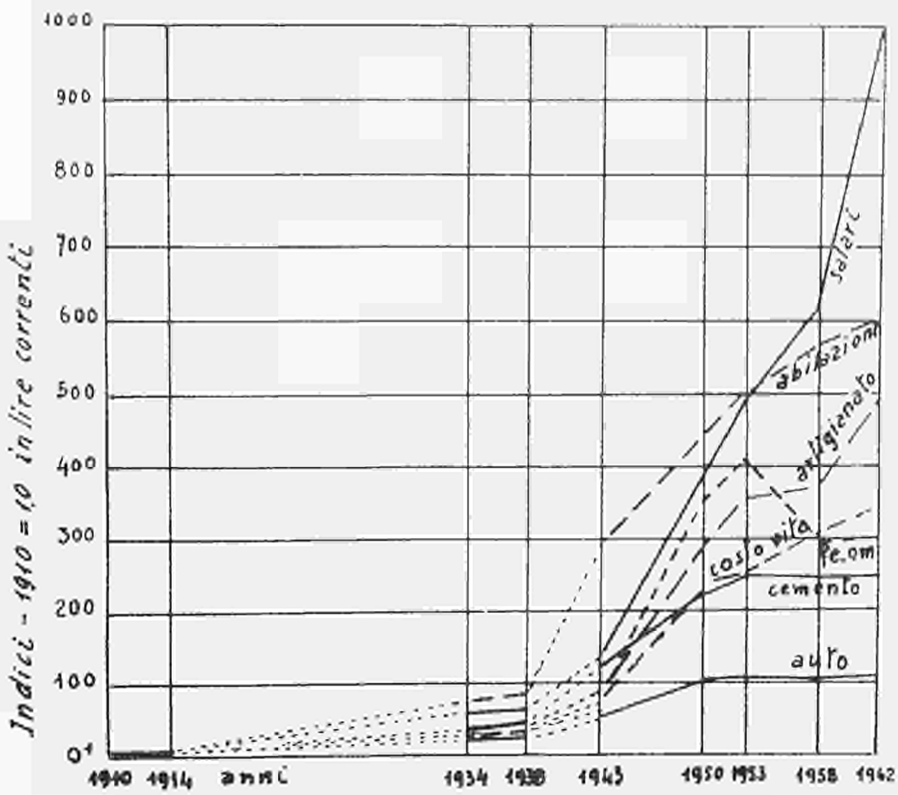

Fig. 1 — Gradient of costs in Northern Italy from 1910 to 1962

Table I

Percentage increase or decrease in the cost of the items of production relating to house building in Italy from 1948 to 1958.

Items	Percentage increase or decrease
Labour	+ 90
Materials	+ 9
Foundations	+ 9
Bricks	+ 30
Covering materials	+ 35
Timber and works in timber	+ 34
Metals and works in metals	− 22
Glass and glass products	− 22
Sanitary installations	− 21
Heating installations	− 22
Electrical installations	+ 16
Paints and varnishes	− 40
Various other items	+ 130
Sum total	+ 35

The very strong influence of the E.C.S.C. is to be noted, because by decreasing and stabilizing the cost of steel it has had an influence on the decrease in the cost of the goods whose manufacture demand it.

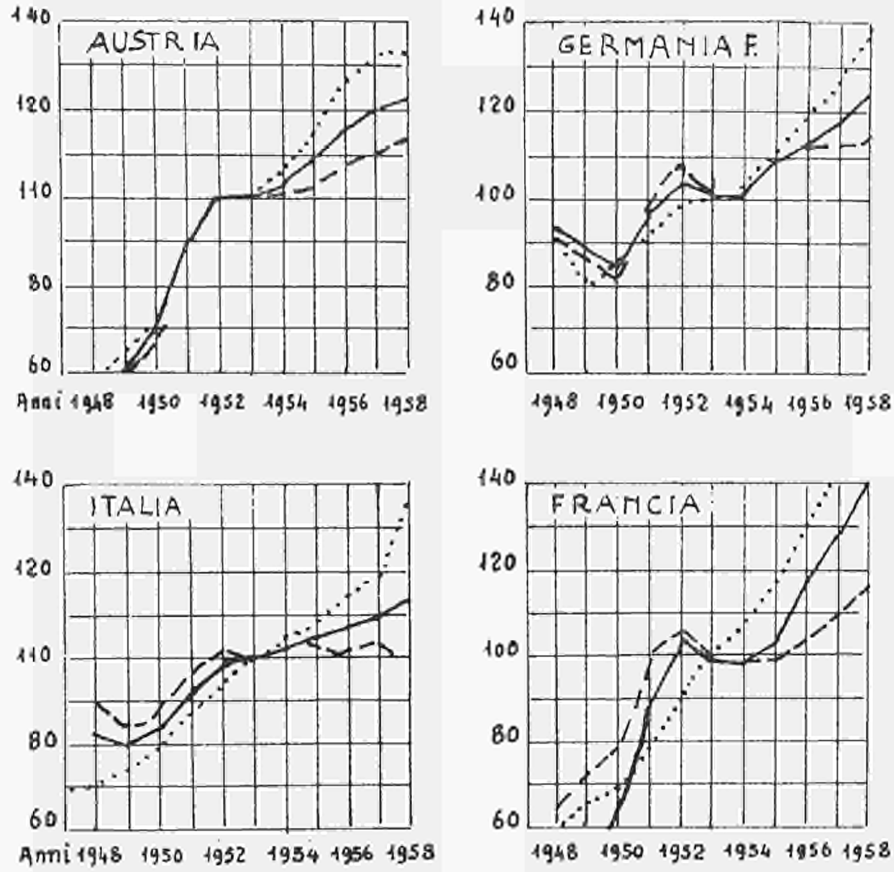

Fig. 2 — Course of the indices of costs in time :
— cost of housing; - - - cost of materials; . . . cost of labour

This is due to various causes, chiefly to the greater demand for comfort, which means greater expense, and partly to the nature of the demand, which up to now has been exclusively individual, and finally, to an extent which is certainly not negligible, the organization of the work, which in many cases has remained of a definitely artisan type.

This is indicated, in any case, from the number of people employed by most of the European building contractors; as shown in Fig. 3, between 57% (in the most favourable case—Federal Germany) and 95% (the other extreme,—Norway) of the European building firms who do not employ more than 10 people.

These firms undoubtedly find it impossible to make a study of costs and programmes and to rationalize production in a manner which the manufacturing industries that wish to survive, consider essential.

It remains to be seen how much this depends on the firms themselves and how much on the system of the building market.

If however, we are to achieve in residential construction the economic progress which mass production elsewhere has achieved, it is essential that there should be a profound change in the methods of marketing, design, construction and erection so as to eliminate all the present drawbacks, namely: very small orders, small, delayed and unreliable supplies, frequent interruptions owing to climatic conditions, faults in design or lack of materials or even of finance, imperfect and incomplete projects, modifications, corrections and additions during the execution of the works, lack or imperfection of programmes and controls on progress, times and cost of execution, irrational utilization of labour force, which is becoming still more scarce and still less satisfied with the bad working conditions on the building site, and with the dependence on climatic conditions.

It is necessary that the house, as with the motor car, that whilst bearing in mind the requirements of appearance, should be constructed by the mass production method, *i.e.* in the workshops, so that a quick mechanized assembly could be carried out on the site.

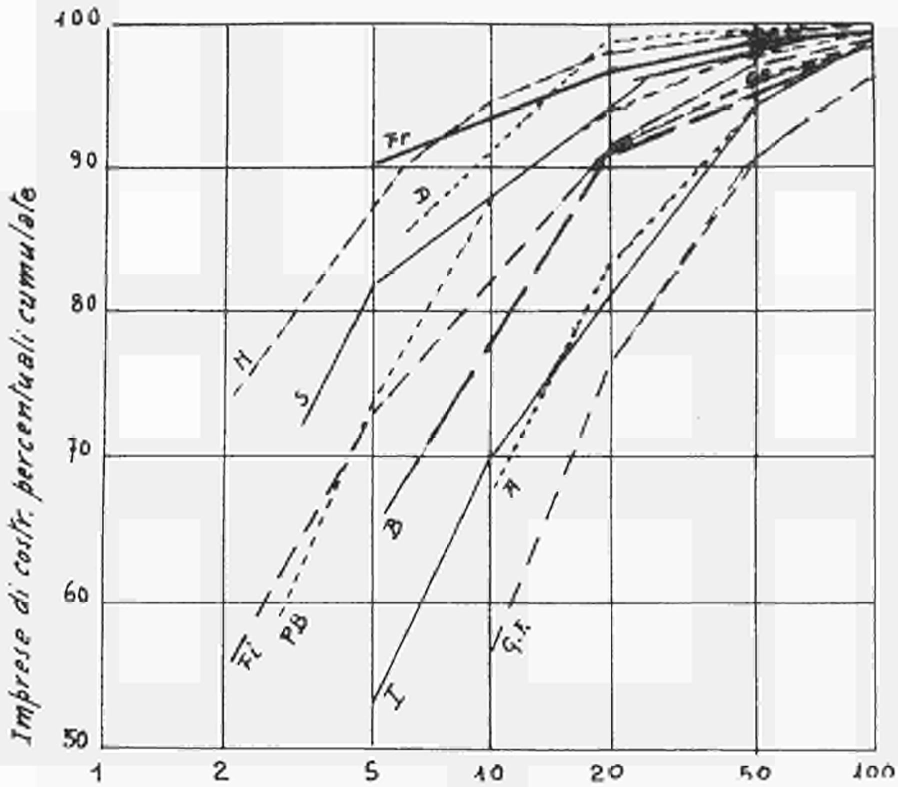

Fig. 3 — Number of employees in construction firms
A Austria – B Belgium – D Denmark – Fi Finland – Fr France – G.B. Great Britain
G.F. Federal Germany – I Italy – N Norway – PB Netherlands – S Switzerland

All this can be very easily achieved, and would be easier than with the traditional building structures, by using steel frameworks. This has been proved by the time taken in erecting skyscrapers for which, in view of the money involved and the size of the construction, it has been possible to prepare efficient methods of procurement of supplies, programmes and controls.

If from residential building we now go over to the building industry who deal with large contracts in which structural steelwork indicates greater possibilities than those of any other building system, namely large bridges, viaducts, flyovers, and certain port and river works, the importance of the contract always permits a more thorough general study. Due account, however, is not always taken of the connection existing between design, workshop construction, transport and erection.

Fundamental unity of productive factors

The study, design and organization of the site for handling the erection of steelwork construction are intimately bound up with the functional study of the structures and their working project on which depend the constructional details, the system of work, delivery times and estimated costs.

Consequently, the factors which greatly influence the most important results, namely, costs and completion time, are found partly away from and partly on the site itself.

Factors outside the building yard

Interdependence between working project, processing and erection

In general, the working project is developed in several stages:

— During the first stage, the functional features of the project are outlined: *i.e.* space, loads, actual weights, and the general scheme of the structure;

— The second stage covers the working project which, whilst observing the functional requirements, makes some essential selections concerning: materials to be used *i.e.* (type of steel: whether it is standard, high-strength, high-yield point, or stainless, etc.). Also the necessary steel sections whether (standard, light or pressed from cold-rolled strip). Then come methods of connection (riveting, welding, hinging, fixing), identification of groups and sub-groups already connected at the works, and methods of erection.

From this study, we derive : first, a working basis necessary for checking the estimated costs and for the design details and subsequently, the details themselves of each element.

— The third stage covers the reconstitution, on the basis of the details, of the sub-groups, groups and the completion which, if well arranged, results in a complete harmony of the total assembly, as well as the exact detailed connection of the various parts (foundations, posts, trusses, beams, stiffeners), and the necessary accessories for the various installations (sanitary, rain, lifting, inspection, etc.)

— Finally the study ends with the working programme which sets the delivery times and justifies the estimated costs and the selection of the various elements which influence times and costs.

From the point of view of efficiency of erection, the accuracy of the details is of extreme importance, a requirement which is just as important as the possibility of interchanging parts in mass production engineering.

Any operation not envisaged in the working project which concerns finish of work, *i.e.* execution of extra parts or some details ommitted or badly made, any variations in the project owing to additions, improvements, finishing touches, involving stoppages or delays which seriously influence the time and cost of erection as well as the success of the whole operation.

Importance of production times — Contractor's costs and buyer's costs

The cost of the construction depends, more than is realized, on the time taken in its execution. Therefore at this point it is necessary to distinguish between the two costs, *i.e.* the contractor's and the buyer's.

Contractor's cost c_i — At first sight we can distinguish, from the various costs of the contrator, two groups, the first produces a value r_1 which decreases as the time taken for construction increases. For instance, owing to the possibility of using less elaborate machines, skilled labour, or of obtaining keener prices from the suppliers, provided, they are not overdone or the times of delivery are not strictly enforced and also possible savings in overtime, etc.

The second group, on the other hand, gives the value r_2 which increases with the time of construction. This class includes: expenses for the service of the capital engaged for a longer period, for the longer period of maintenance of the site, use of the plant and for higher overhead expenses.

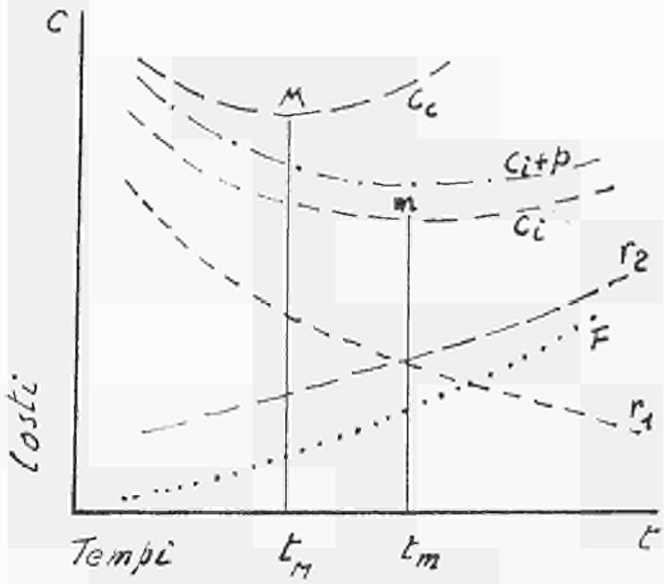

Fig. 4 — Contractor's and buyer's costs

The sum of the two values $c_i = r_1 + r_2$ gives the curve c_i (Fig. 4) representing the contractor's cost, which evidently shows a minimum at the point m, corresponding to the time t_m.

Buyer's cost C_e — At first sight, the buyer's cost appears to be of the type

$$C_e = c_i \pm p$$

that is, consisting of the contractor's cost c_i plus the corresponding profit p which can be positive or negative.

In reality this is not true and the difference is greater because even for the purchasers cost there is a financial outlay F which takes into account, among other things:

(a) the interest on the money used in the cost of construction up to the moment when it is used;

(b) the consumers overhead expenses incurred in order to follow up the work from the administrative and technical points of view (Government and municipal formalities, surveyor, control of materials and work);

(c) lack of anticipated profit from the complete operation which can only commence from the commencement of the operation. Consequently, a delay in delivery involves a delay in creating profits and even perhaps a decrease in same. For dwellings, owing to a larger offer which can take place in the meantime and which offsets the demand, and for industrial buildings owing to a reversal in the market).

The previous equation, therefore, is to be altered as follows:

$$C_e = c_i \pm p + F$$

If on the diagram shown in Fig. 4 we plot the growing line F, and after marking a reasonable increase (about 10%) of the cost of the contractor c_i to allow for the profit, if we take the sum in order to obtain the curve of cost C_e, we have in general another minimum value of M, giving a construction time t_M which is lower than t_m.

This means that the minimum costs of the contractor and the consumer do not generally coincide and that the consumer may find it convenient to accept a higher cost and, consequently, a higher price from the contractor in order to reduce his own cost to a minimum.

This is often done in an approximate way by laying down a premium in the case of an advanced delivery and a penalty in case of delayed delivery.

On the other hand, the cost of the money tied up constitutes a charge for the contractor whose disbursements always precede remittances from the consumer to the contractor. This cost is a minimum at the commencement of the work because little has been done, but it increases as the construction proceeds because to the cost of materials, almost entirely used, are to be added the cost of labour, transport, overhead expenses and unforeseen items.

It is consequently most important to expedite progress of work as it reaches the most advanced stages and sometimes it is possible in steelwork constructions to arrange for the fabrication in the workshops and the erection on the site to proceed at the same pace.

In Italy, for instance, during recent years, preference has been given to steel structures in the case of all industrial buildings, owing to the possibility of obtaining in 3-4 months building-sites having an area of 100,000 sq. metres and over, whilst in the case of reinforced concrete structures, the time required wloud have been at least two years.

For an accurate study of the possible coverage for the various stages of work in the factory and at the place of erection, the classical methods of Gantt and Adamiecki are very useful and to these, in the last few years, a very efficient though more complex method has been added, namely the Pert.*

Interdependence between project, construction, transport and erection

For the Contractor, the cost of a construction may be deemed to consist of the following elements:

$$c_i = m_{ac} m_{oc} s_c P + m_{at} m_{ot} m_{st} P + m_{am} m_{om} m_{sm} P,$$

in which

m_{ac}, m_{at}, m_{am} are coefficients depending on the outlay for materials used for the construction, transport and erection;

m_{oc}, m_{ot}, m_{om} are coefficients depending on the cost of labour for the construction, transport and erection;

s_c is a coefficient depending on the overhead expenses relating to the entire construction;

m_{st}, m_{sm} are coefficients depending on the special overhead expenses relating to transport and erection;

P is the total weight of the construction to be made.

If we call:

$$m_{ac} m_{oc} s_c P = m_1 P\ ; \qquad m_{at} m_{ot} m_{st} P = m_2 P\ ; \qquad m_{am} m_{om} m_{sm} P = m_3 P$$

we have:

$$m_{ac} = \frac{C_i - P(m_2 + m_3)}{P(m_{oc}\, s_c)}\ ; \qquad m_{oc} = \frac{C_i - P(m_2 + m_3)}{P(m_{ac}\, s_c)}\ ; \qquad s_c = \frac{C_i - P(m_1 + m_2)}{P(m_{ac}\, m_{oc})}\ ;$$

$$m_{at} = \frac{C_i - P(m_1 + m_3)}{P(m_{ot}\, m_{st})}\ ; \qquad m_{ot} = \frac{C_i - P(m_1 + m_3)}{P(m_{at}\, m_{st})}\ ; \qquad m_{st} = \frac{C_i - P(m_1 + m_3)}{P(m_{at}\, m_{ot})}\ ;$$

$$m_{am} = \frac{C_i - P(m_1 + m_2)}{P(m_{om}\, m_{sm})}\ ; \qquad m_{om} = \frac{C_i - P(m_1 + m_2)}{P(m_{am}\, m_{sm})}\ ; \qquad m_{sm} = \frac{C_i - P(m_1 + m_2)}{P(m_{am}\, m_{om})}\ .$$

It is now possible to pass to the *confluential* analysis in order to establish the independent linear ratios between the 10 variables, evaluating statistically those ratios so as to make c_i as low as possible.

This research is complex but is worth studying in the case of large constructional works such as skyscrapers, aircraft hangars, and complete residential estates, whilst remaining simple and interdependent. For normal constructions, however, this method would be unduly onerous and generally speaking an accurate analysis carried out as explained later would be more than sufficient.

As the construction always constitutes by far the largest cost, we shall commence by determining the various constructional systems which might be advisable, and fix their costs.

Then, for each of the constructional systems deemed to be worthy of consideration, we shall examine the various possible solutions with reference to the influences they may have on the cost and duration of transport and erection and so rate the financial values.

In this connection, we must take special account of the risk due to climatic conditions, which varie considerably according to the method of erection selected, as follows,

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* Programme Evaluation and Review Technique.

(a) Constructions which allow for a very rapid erection, with powerful mechanism, also where the labour force is not unduly exposed to the discomforts of the climate.

(b) Having moderate unforeseen charges, and consequently a limited risk, as compared to erections involving the laying of single bars, one at a time, heavy riveting operations or welding work, showing considerable risk as regards cost and loss of time.

Having experimented with a Gantt diagram the possibility of overlapping the various stages and, consequently, the times estimated for their execution, we can now prepare a table of the type shown in figure 5.

System of construction	Realization of groups	Manufacture		Transport		Erection		Total time	Economic evaluation		Total cost
		time	cost	time	cost	time	cost		time	risk	
1	2	3	4	5	6	7	8	9	10	11	4+6+8+10+11
A	1										
	2										
	3										
	4										
B	1										
	2										
	3										
	4										
C	1										
	2										

Fig. 5 — Table for determining the best method

Of the many possibilities selected which in one form or another must be used during the project, I will recall a few as follows:

(a) With regard to the form of structure, it is necessary to establish whether it is advisable to adopt the reticular classical fully isostatic type, or the hyperstatic type, without rods, but with frames,

(b) or the full-wall type. In respect of either case, the weights of materials, hours of work and cost of transport and erection change, must be considered.

(c) With regard to the elements of the structure, whether to give preference to identical interchangeable types, as in the case of the Cadiz Towers, which allows for the in-line automatic workshop construction, but considerably increases the time for erection,

(d) To prefer instead, types of variable structures which do not require in-line automatic workshop construction.

(e) As regards joints, whether to use rivets, electric arc welding or high-strength friction grip bolts, either in the workshop or on the site.

(f) As regards the space occupied by the various parts delivered to the site for erection, *i.e.* whether to fabricate in the workshop heavy and cumbersome assemblies which can be lifted and installed using heavy gear, equipment or whether to adopt an intermediate solution by fabricating in the workshop medium-length parts of average weight which can be connected together on the site. For example using the available space on site so as to proceed thereafter with the remainder of the erection, similar to the erection of bulkheads in shipyards, or they can be erected directly on the site using small mobile cranes.

(g) With regard to the handling plant, whether we should use cranes mounted on rubber tyred wheels or on rails or even self-lifting cranes connected to the structure. For example, in the case of bridges, the handling plant can be positioned along part of the river bed or on the abutments, which are situated on the bank. This plant can be of a mobile or stationary type, fixed to the ground or floating and anchored to special wire ropes or to the suspension cables of the bridge, etc. Thus we go from gantry cranes with a span of 36 metres and a lift of 800 tons (similar to the one on rails for the Cologne-Deutz bridge), to pontoons equipped for lifting complete beams being transported by river.

For the framing of buildings, which are the main issue of this subject, the selections to be made will be discussed in the chapter specially devoted to work on the site.

It is not unusual, however, that when dealing with residential buildings, to mention handling equipment which is used for the erection of other steel structures; in fact, not only are the fundamental criteria which inspire the technology of erection always the same, but often there is a strict functional correspondance between the methods which at first sight might seem typical of certain constructions. Consequently, the methods used for solving these problems may suggest useful applications in entirely different fields.

It is obvious that each of the decisions briefly referred to above, influences in various ways, as already stated, other operations of the cycle, namely, the procurement of supplies, the construction itself, transport, erection, times of execution and finally the individual and total costs.

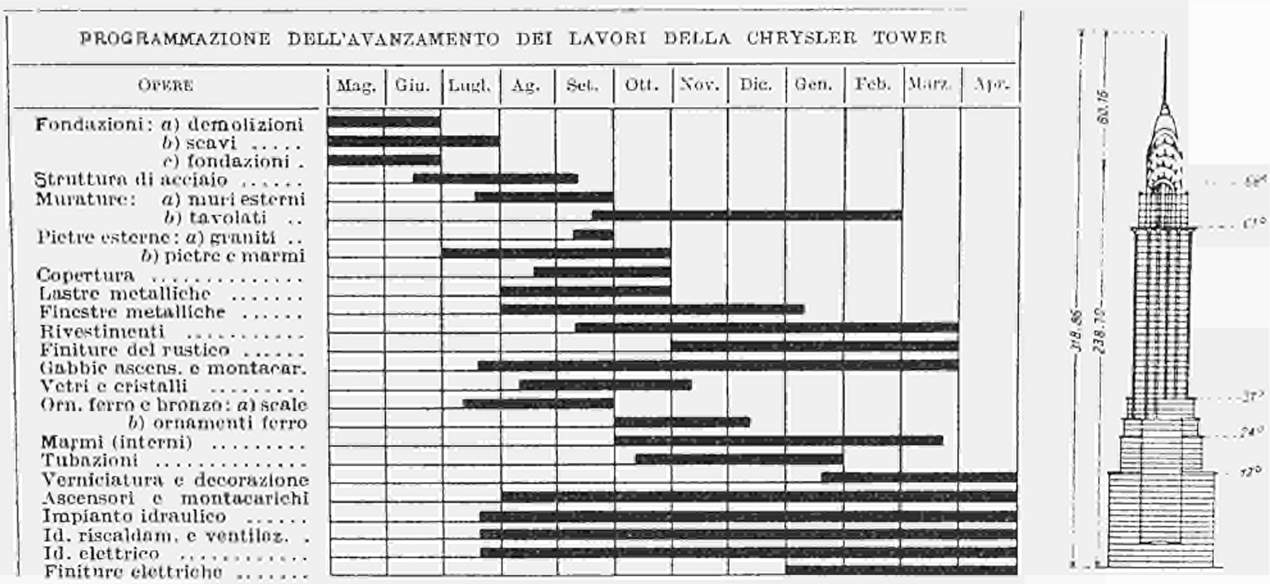

Fig. 6 — Gantt's diagram for programming the works of the Chrysler Tower

Programme of the works progress of the Chrysler Tower Works

Works

Foundations : (a) demolitions
(b) excavations
(c) foundations

Steel structure
Walling : (a) external walls
(b) partitions

External stones : (a) granite
(b) stones and marble
Coverings (roofing)
Metal plates
Metal windows
Coverings
Finish of rough work

Lift and elevator cages
Glass and crystal
Architectural iron and bronze :
(a) staircases, (b) ornamental iron
Marble (interior)
Piping
Painting and decorating
Lifts and elevators
Sanitary installation
Heating and ventilating installation
Electrical installation
Electrical fittings

An operation may, for example, increase the cost of the materials, whilst reducing the cost of labour. It may require more onerous handling equipment whilst reduring the times of erection and delivery, etc.

The study of the technical and economical interdependences of the various representative coefficients allows us to solve, as we have seen, the problem of the optimation of the total costs, either for the contractor, or for the consumer, or both.

With regard to the two methods of investigation previously mentioned which suggest reasonable and rational decisions, we find that they correspond, in general, barring differences of detail, to those used in the principal specialized research work in their own field, and apply equally well to the study of steel structures in buildings.

We note, however, that they approach closely to the British Standard Regulations concerning the analysis of costs in residential building construction.

The method of checking the estimated costs of a project, as advised by the British Committee for Cost Study, [1] is an operative research carried out in two stages. The first studies the influence on the costs of architectural town-planning decisions concerning various ways of the arrangement of the land and the buildings to be erected on it. The British Ministry of Housing has published Regulations [2] in connection therewith. Calculations are made first of a prime general cost and then a detailed analysis is made of the elements and methods of the construction, considering with care those methods which have a greater bearing on costs. This is in order to find possible alternatives which may reduce them, and at the same time materials used, and examining the methods envisaged for their assembly.

The second method, more empirical, proposed and used with good results by the Ministry of Education in order to realize the School Building Programme, approaches in general the lines of the one previously proposed for buildings not having a larger size.[3]

Factors inside the building site

Since, as we have seen, there are important interdependencies between the operations preceding the erection and the specific ones referring to the erection proper, and that they may bear heavily on costs and times of delivery, it is desirable that the final project should include the entire study of the construction and should also exactly define the erection operations and the mode of execution.

Fundamental consideration. It must be remembered that in many cases the greatest advantage of the steel construction is the possibility of completing it very rapidly.

This is the feature which, for large industrial buildings, makes it at the moment preferable to any other constructional system.

The designer and the contractor who do not avail themselves of this foremost advantage, neglect a source of great financial gain and hinder the development of steelwork construction. This is desirable in order to permit the steel industry to make continuous progress in quality and cost and also to permit an ever greater and quicker development of suburban class and council dwellings for the benefit of the people.*)

Structural steel buildings were raised under the banner of speed.

In 1861, Paxton had erected in London, in six months, the Crystal Palace covering an area of 98,000 sq.m. using 9,642 tons of ferrous materials and, since steel sections were then not in existence, he had to use cast-iron for the rods subjected to compression.

In 1889, Eiffel had erected in 25 months his tower of 300 metres weighing about 7,000 tons and comprising 12,000 elements connected by 2,500,000 rivets weighing 450 tons.

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[1]) I will mention, as an example, a large multi-storey office building in the course of construction which, judging by the way the work is going, will require at least 2 years or more before it is completely erected and put into service.
The value of the building can be estimated at about 1,000 million Lire, the value of the land on which it stands is 1,500 million Lire. The cost of the steel structure will be about 400 million Lire. Judging from what has happened in the case of some American skyscrapers (Fig. 6) and the Palace of Nations in Paris (Fig. 7) as well as the new Fiat buildings in Turin, the construction could have been completed in six months, approximately.
On account of interest on capital alone (which to-day in Italy stands at about 10%), the constructor of the frame has lost about 30 million Lire which could have been about the net profit for his work, but the owner, in respect of interest alone, has lost 15% of the value of the land—in round figures 200 million Lire—and about 10% of the value of the building—in round figures 300 million Lire—namely, 30% of the value of the building without taking into account the lower presentday value of the offices for which the market demand is lower to-day.

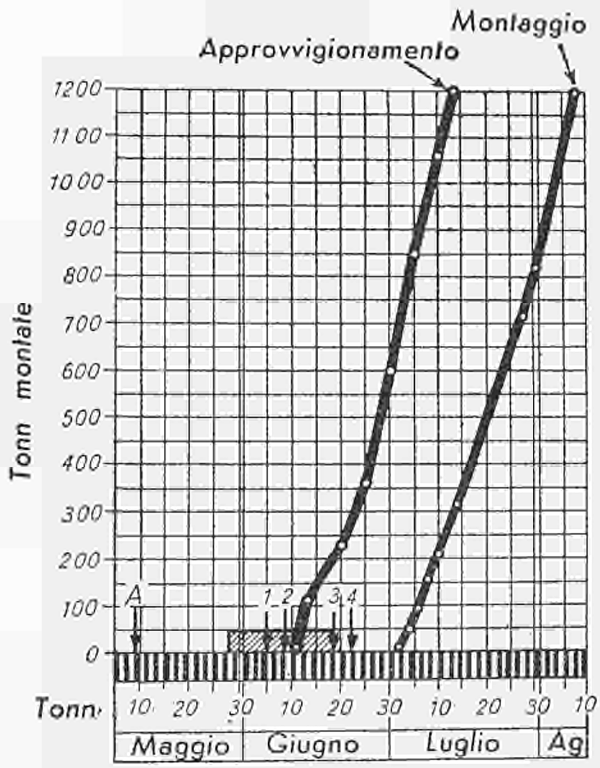


Fig. 7 — Construction programme of the Palace of the United Nations in Paris. A) Commencement of project : 1. Delivery of first lot of rolled steel ; 2. Delivery of second lot ; 3. Delivery of third lot ; 4. End of rolling and delivery of balance of rolled sections. For the erection, 18 fitters erected 25 tons of frame per day (1,400 kg. per day per fitter)

In view of the foregoing, it is obviously convenient to consider the erection in combination with the design and the production in the workshop, as is normally done in the case of large steel bridges.

On the other hand, those who have the task of organizing site work and directing it in accordance with the data of the project, must have at their disposal all the information necessary, as envisaged by the designer in relation to the erection, This covers the weight and shape of the components of the structure, their strength and maximum stresses expected during the erection, systems of provisional and final connections handling plant to be used, time of delivery of the various parts and progress schedule of the works. Unfortunately, this is not always the case and only times and costs of erection are set out, estimated on the basis of previous experience. It is left to those in charge of erection, who are often sub-contractors or external piece-work operators, to select and procure the handling plant, to prepare the site and all necessary operations.

Undoubtedly those responsible for the erection must have the freedom to make a closer study of the matter and to propose alternatives which they may deem useful, but this should always be carried out in full agreement with the designer.

However, on the basis of the supplied data, the Agent should prepare a very accurate and detailed programme in accordance with the available handling equipment, also the time limit and the money allocated and should obtain all outstanding data not yet made available to him by the Design Office and the Resident Engineer.

The programme, however, cannot be considered to be rigid, since many events may influence the development of the operations.

Among them, of predominant importance, often due to climatic conditions are delays in delivering, handling plant or various parts of the construction, also work stoppages due to strikes or other causes of "force majeure".

For this reason, from the commencement, the programme must allow for the most probable events and have ready the remedies to be put in operation at the right moment in order to decrease their effects.

For instance, in the case of bridges and quay installations great importance is attached to the conditions of the rivers or the sea, and to the possibility of floods and inundations. Therefore it is essential to make an accurate study of statistics for at least the previous 50 years in order to be aware of all the possible conditions.

The programme should be based on probable averages, but capable of utilizing the best conditions, if they should be present, or the worst if, through bad luck, they should be encountered.

Requirements to take into account when planning the erection

(1) In the first place the safety of those in charge, and in the second place good working and living conditions, wherever possible.

(2) Safety of the machinery and the construction in relation to wind (collapse), ice (welding), avalanches landslides, floods and inundations. If the risk of sea storms or floods is expected, the machinery must be placed in a safe position (for example, in the case of bridges, on the abutments) or must be capable of being quickly withdrawn (planning of caterpillar cranes and safety roads).

(3) The maximum use of the handling equipment and personnel so as to keep both fully employed as far as possible.

(4) Harmonizing the use of all the equipment so that there should be no interference or disturbance.

Planning the erection

Study of the cycle. The fundamental study, closely bound up with the design is the definition of the erection cycle which involves the selection of the method used and fixing the sequence of operations and corresponding times.

Study of the method. This is of extreme importance. More than just saving labour, and is obtained by reducing the working hours, therefore planner must and can rely on the saving obtained by choosing the best method.

In some cases, by studying the method, very notable savings have been obtained.

The times usually allowed to-day for the erection of the normal structures such as large sheds or industrial establishments are as follows:

— for steel structures with simple frames and equal sections, weighing more than 50 kg./sq.m. and of a total weight of at least 500 tons	30-35 kg.	per man-hour
— for similar structures weighing less than 50 kg./sq.m.	28-32 kg.	do.
— for similar structures, but with complex, non-standard frames and of a total weight lower than 250 tons	25-28 kg.	do.
— for more complex structures also comprising mechanical equipment (cranes)	15-20 kg.	do.

As shown in Fig. 7, for the Palace of Nations, a productivity of 140 kg. per man-hour was reached. Even allowing for the notable weight of the structure and the exceptional handling equipment which were otherwise a charge on the cost, the savings achieved were exceptional.

It is impossible to check beforehand, by direct investigation, whether the pre-set times are reliable, before the application of the pre-selected method. Therefore importance must be placed on the evaluation methods based on the elementary times of the M.T.M. type and the like, suitably simplified.

Figure 8 shows an application to a normal working squad.

Fig. 8 — Study of times for teamwork

With this system, it is possible to make a rapid comparison between different solutions and select the one which appears most satisfactory.

In order to determine the influence which the method of study may have on the other stages of the construction and on the work progress, Gantt's diagram is very useful and an example of it has already been given in Fig. 7 (actual check) whilst Fig. 9 gives another example (estimated study).

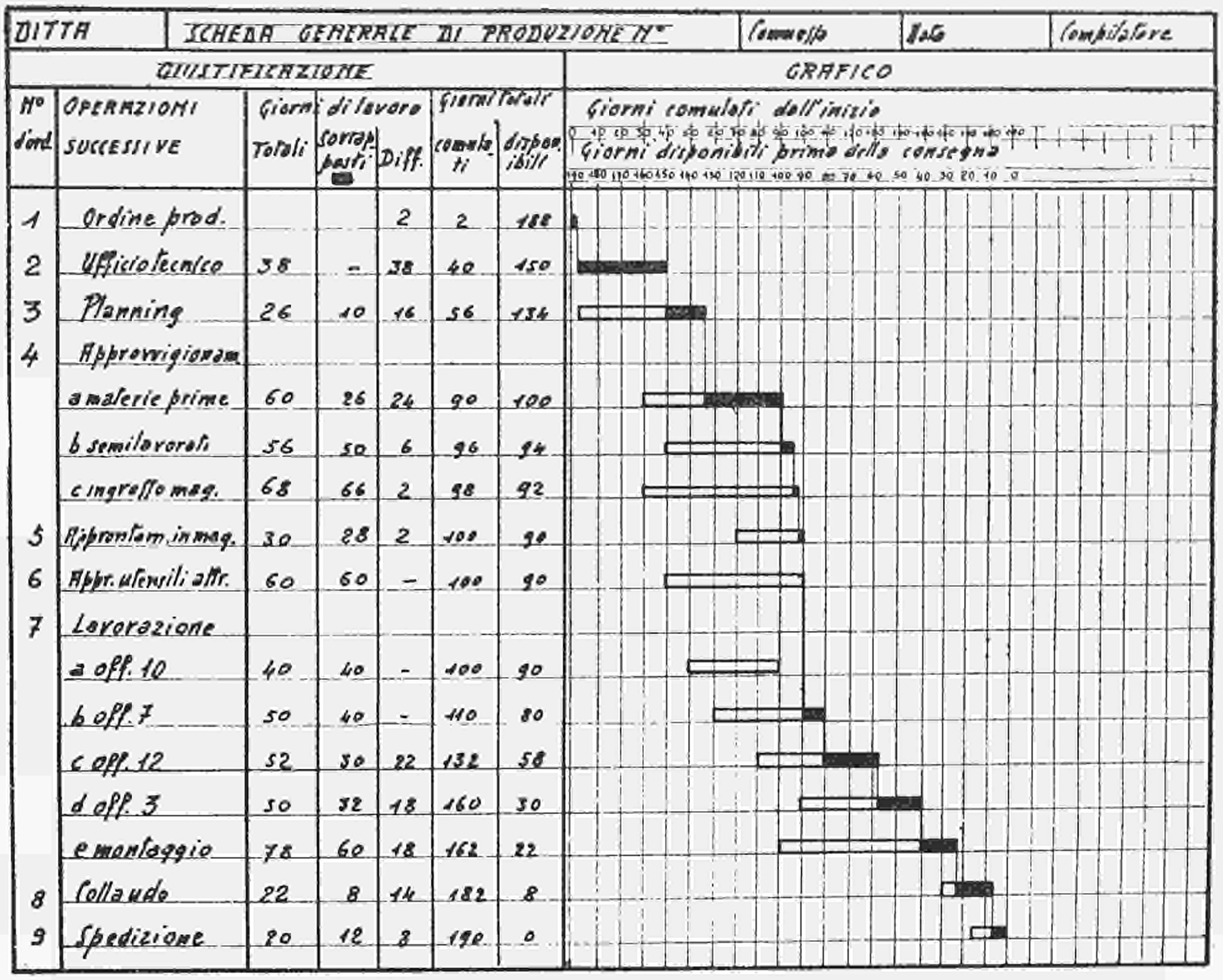


Fig. 9 — General production diagram (Gantt) — Programme

Firm. General production card. Order. Date. Verified.
Justification — Graph
Consequent operations — Working days — Total days
Days accumulated from the commencement — Available days before delivery

1) Production order
2) Technical office
3) Planning
4) Procurement of materials
   a) raw material
   b) semi-finished
   c) warehouse (entries)
5) Stocking in warehouse
6) Preparing tools
7) Work
   a) workshop 10
   b) workshop 7
   c) workshop 12
   d) workshop 3
   e) assembly
8) Test
9) Dispatch

More details are given in the working charts (Fig. 10).

Gantt's diagram is also used for following up the work, checking the correspondance between times and estimated costs with the actual ones and for identifying the reasons for any discrepancies or dangers or of unforseen conditions so as to offset the consequences by means of immediate decisions. (Fig. 11).

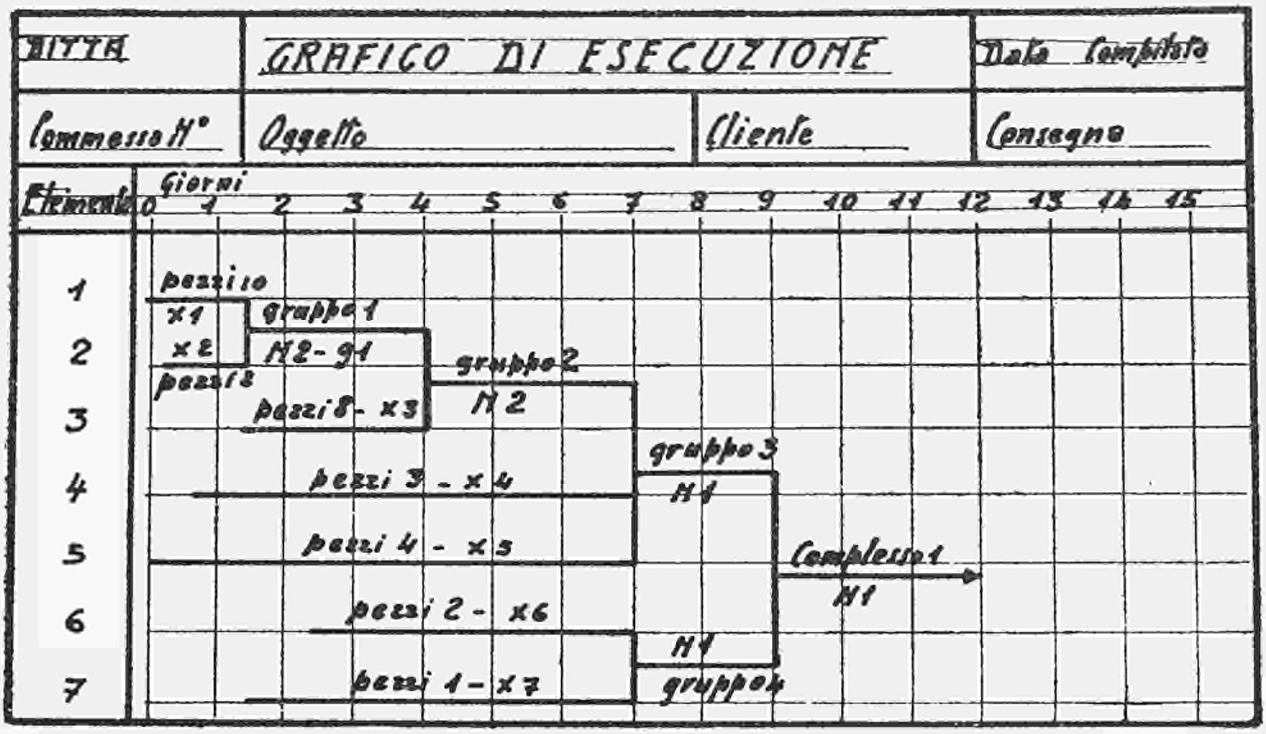


Fig. 10 — Execution graph for the erection of a section of elevated road

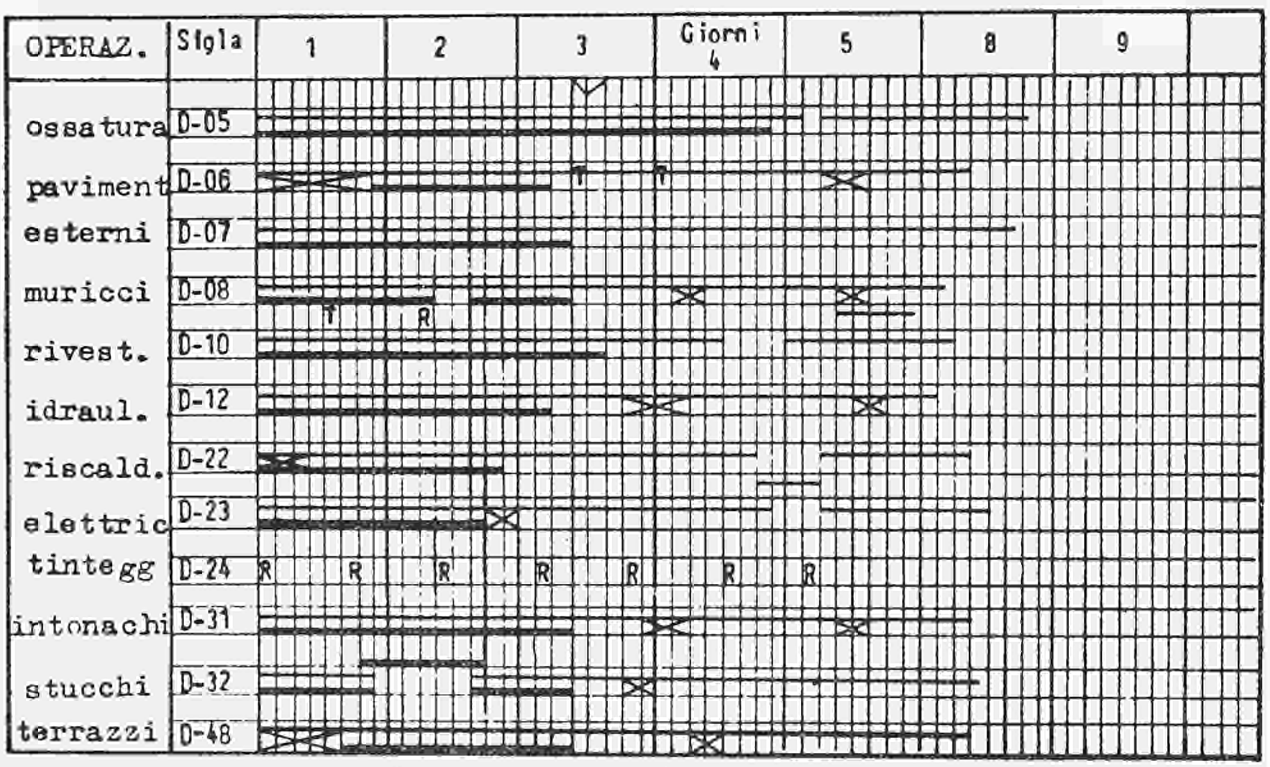


Fig. 11 — Checking the progress of the works with Gantt's diagram

Frame
Floors
Outside

Small walls
Coverings
Sanitary installation

Heating installation
Electrical installation
Water painting

Distempering
Plastering
Terrazzo work

Conventional symbols

= Commencement of a set of works (floor, pile, etc.)

= End of a set of works

= The thin line indicates the estimate of work

= The thick line indicates the actual duration of work

= The day when the programmes are postponed

3722/45 = A given work and the number of units to be carried out

R = Stoppage owing to impossibility to work

T = Stoppage through lack of materials

= Reserve time for completions and finishing

Although Gantt's diagrams, if properly used, can be fully satisfactory, they usually do not take into account, all the events (not being designed for that purpose) such as, orders, expense authorization, allocation of funds, etc. which are very important for the execution of the work from every point of view, and especially with regard to times of execution, and delivery ,also individual and total costs.

The practical realization of these imperfections, especially when programming such large and complex constructions as interplanetary missiles, has given rise to the idea of a more complete and complex system, the Pert, which the U.S.A. Navy realized for the first time in 1959 through the elaboration of the working plan of the Polaris Missile.

The Pert is just a "grafo" (graph)*, based on the working structure of the job to be done in which the best sequence of operations is fixed taking into account: their interdependence, possible overlapping, various activities connecting the events (orders, authorizations, availability of funds, productive operations, etc.) and all the internal and external details which may have an influence on the execution of the work.

Of the various events, special consideration is given to the *critical* ones, the times of which must be rigidly maintained if the times of delivery and the pre-fixed costs are to be respected, whilst the others which allow for a certain flexibility are less important.

In the graph, the circles (knots) indicate the events (stages); each stage represents an increment of realization of the programme. The stages are connected by lines (arcs), their arrows showing the direction of progress. (Fig. 12).

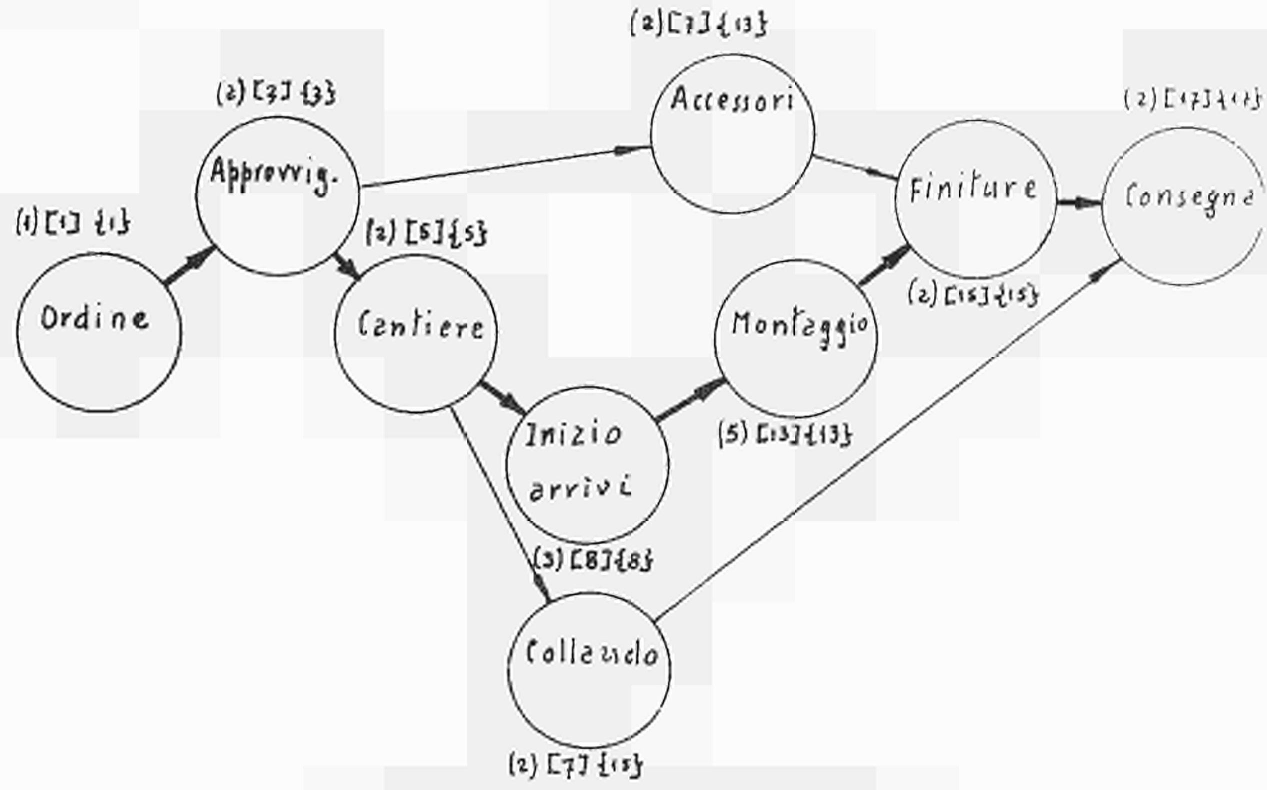


Fig. 12 — Example of "grafo" (graph) for the Pert

* A "grafo" (graph) consists of a unit x (a, b, c, ... n) shown in the drawing by circles (knots) connected to one another by a second unit U of pairs (a, b) (c, d) ... with ax, bx, ... represented by various arrow connections (arcs).
The "grafo" G defined by x and by U is represented by G (x, U).

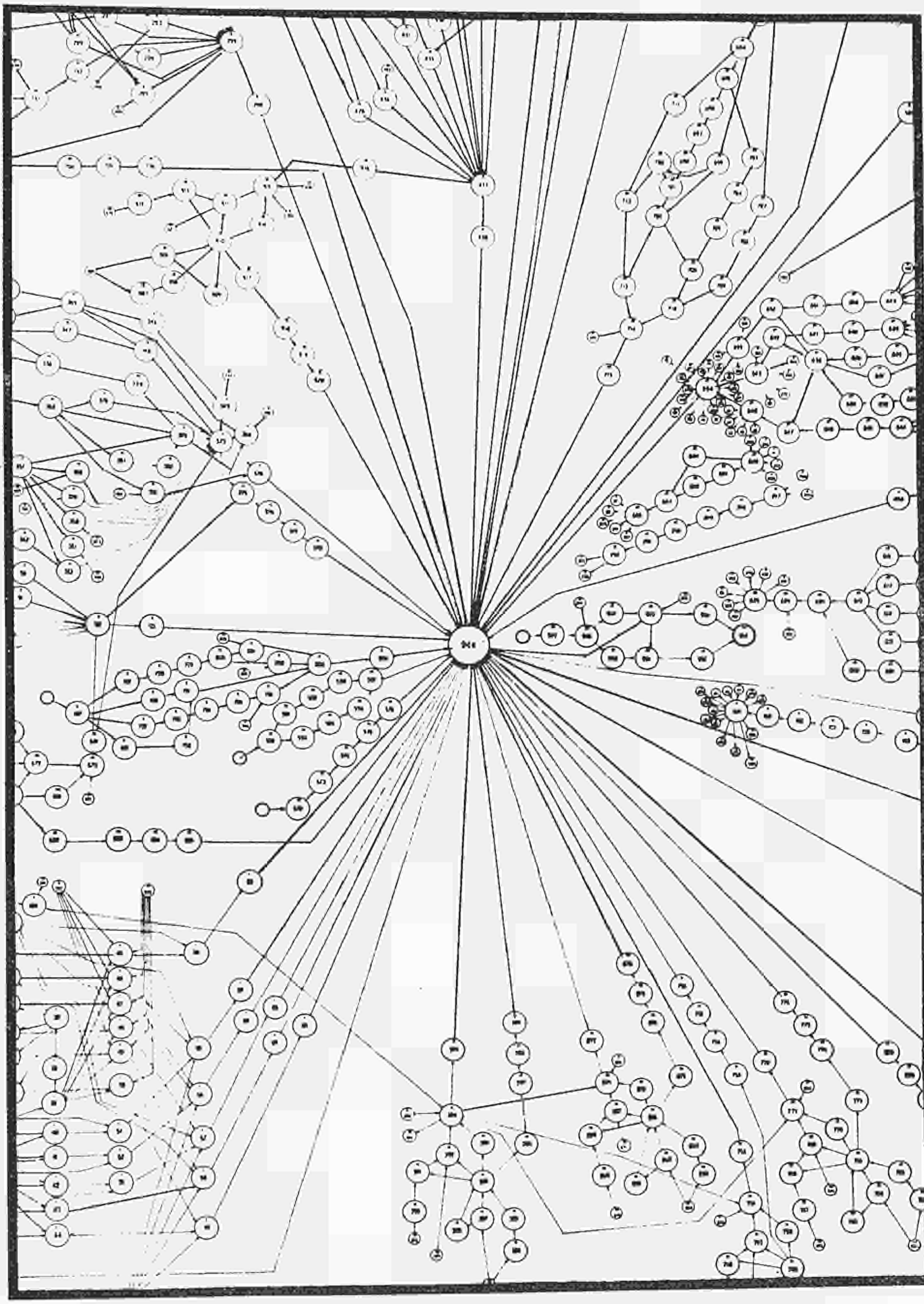


Fig. 13 — Pert-Olivetti diagram for the Zanussi Company
Detail of the Pert graph for programming the construction of the new Washing Machine Factory

All graphs show some circuits in parallel, partial or otherwise; it is thus possible to follow different routes, proceeding from the initial knot, decision to undertake the work, to the final knot (delivery of the work tested). The path connecting the critical knots is a critical path, marked in a heavier line and indicates the stages which must not undergo changes, including the minimum time of delivery.

In Italy, this method has been successfully adopted by many large industrial organizations, for instance Enel, and used for programming and controlling hydro-electric and thermo-electric works. Fig. 13 shows the Pert designed for the installation of an industrial establishment constructed chiefly in steel.

Fundamental methods for the erection of steel structures

It is necessary here to distinguish between the most common types of frames.

Light structures for small sheds, agricultural buidings, platforms and the like.

In the case of these simple constructions, it is not possible to think of highly mechanized installations because they are destined to small owners who often wish to erect them themselves.

In this case, their main feature is the self-erection owing to the extreme simplicity of the parts, already connected as far as possible in the workshop, forming units which can be easily transported and handled (not more than two men per unit) and can be easily connected to one another by hinges. Fittings and bolts fixed to one of the parts so that they cannot fall off and be lost. (Fig. 14).

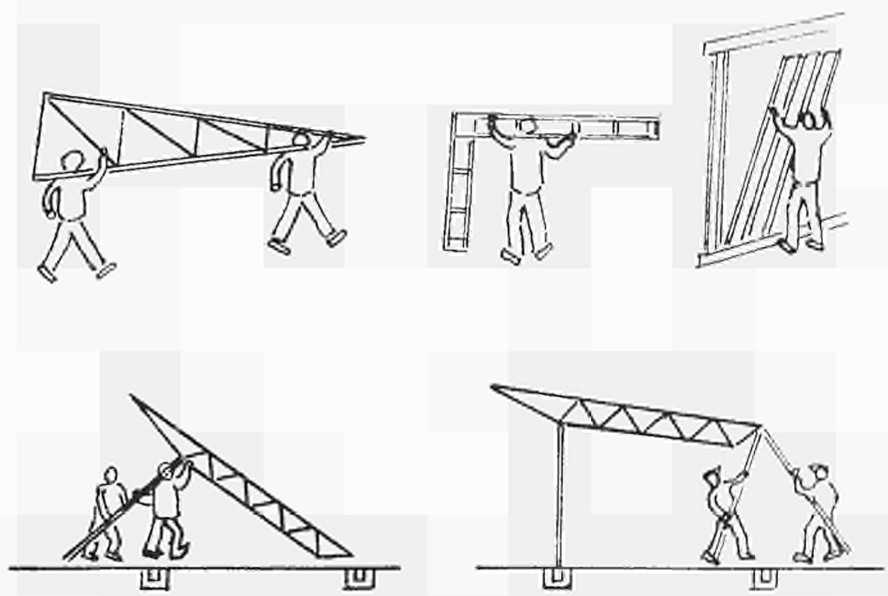


Fig. 14 — Erection of simple structures for agricultural buildings

For these types of constructions, the lower cost must result from simplification, standardization, limited weight and the mass production of elements which can be connected in various ways. It is then possible to put up constructions of different types, plans, features and uses, although always utilizing the same components.

Medium structures for two or three storey buildings, of individual type.

The mechanization must be moderate. The components must have dimensions such as to allow their transport by usual lorries and trailers and also of weights which are compatible with the lifting capacity of an normal crane on rubber tyres. For these cases, self-propelling cranes on tyred wheels with a long jib and rotating boom (Fig. 15a) rightly dominate the market in America.

In Europe, use is often made of the usual tower crane and horizontal rotating jib, which is also used for reinforced concrete constructions, in which case it runs on rails. (Fig. 15b).

For a series of similar buildings, to be erected in line, use is also made with advantage of the gantry crane (Fig. 15c), but often it would be more advantageous to use cableways (blondins) with swinging pylons, which are faster and can use two trucks simultaneously on areas of 500 × 20 metres and, for larger areas, double cableways (blondins) with funicular crosswise connection (Figs. 15d and 15f).

Important structures for industrial sheds.

In this field, the tyred, fast, self-propelling cranes, in sufficient numbers, dominate.

Important structures for buildings with up to 12 storeys.

Up to a few years ago, the system generally used in Europe, for these cases, was based on one or more horizontal jib tower cranes, whilst in the U.S.A. preference was given to the derrick cranes.

At the present, however, there have come into use systems for erecting entire storeys and sides constructed generally on the ground or at a pre-fixed level where the main site is arranged.

The basic types of this system, which has given good results, can be divided into three varieties:

— The first system envisages the construction of a particularly strong central component of the tower type (staircase and lift well) used as a support and guide, along which are placed the entire floors generally constructed on the ground, or sometimes at the top, but in either case at the level where the equipment and the forms for building the floors (Fig. 16a) are in operation.

  The tower can be built as the first floor (the highest, being the flat roof), and rises to reach its own level or can be built by first installing at the top all the hoists for lifting the floors, which can go up altogether in a bundle and be left at their respective levels or by lifting one at a time by hydraulic jacks operating at the bottom.

  In this type of construction, the floors are cantilevered and supported by diagonal ties leading to the tower.

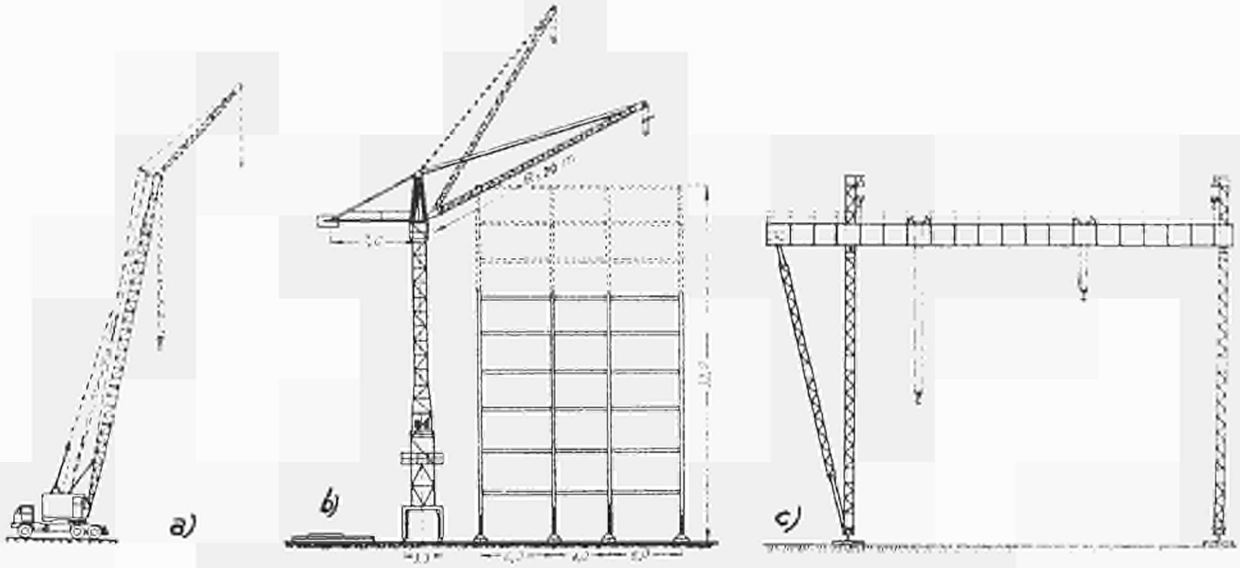


Fig. 15 — Erection cranes — (a) on tyres — (b) tower type on rails — (c) gantry

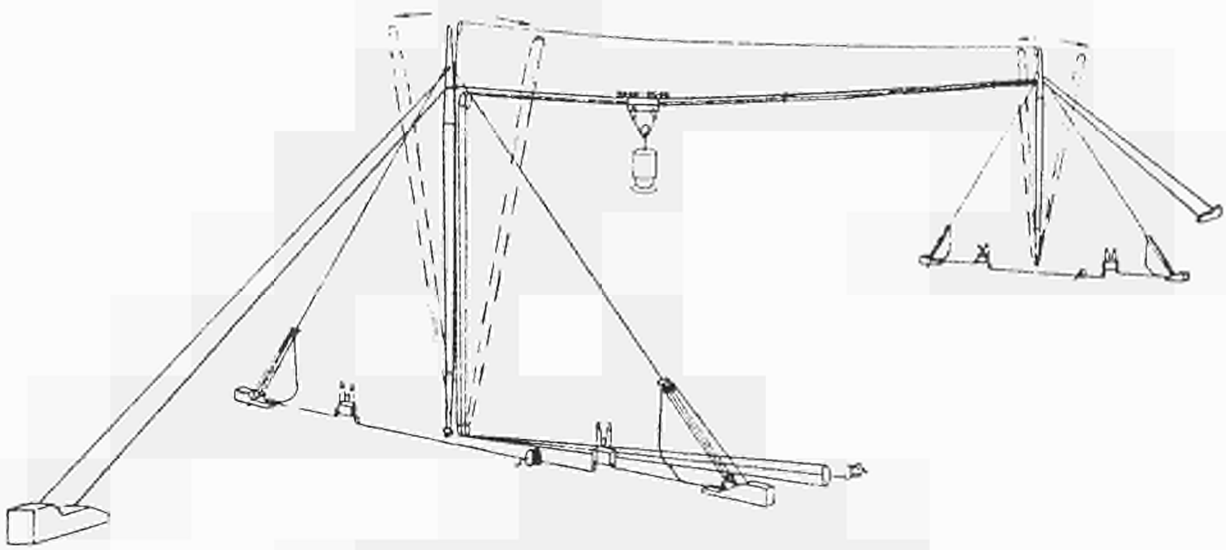


Fig. 15 d — Cableway (blondin) with inclinable pylons

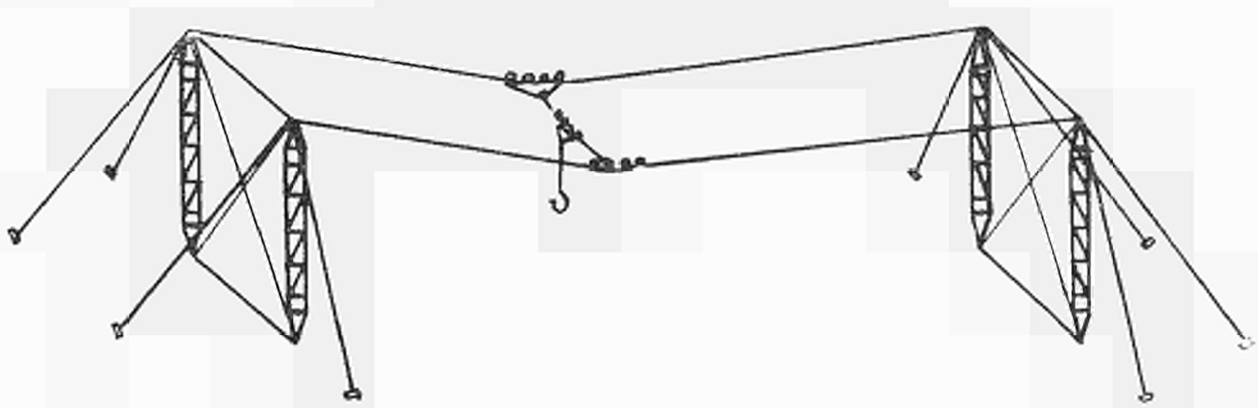


Fig. 15 e — Double cableway (blondin) with cross-carriage. It covers 500 × 30 metres

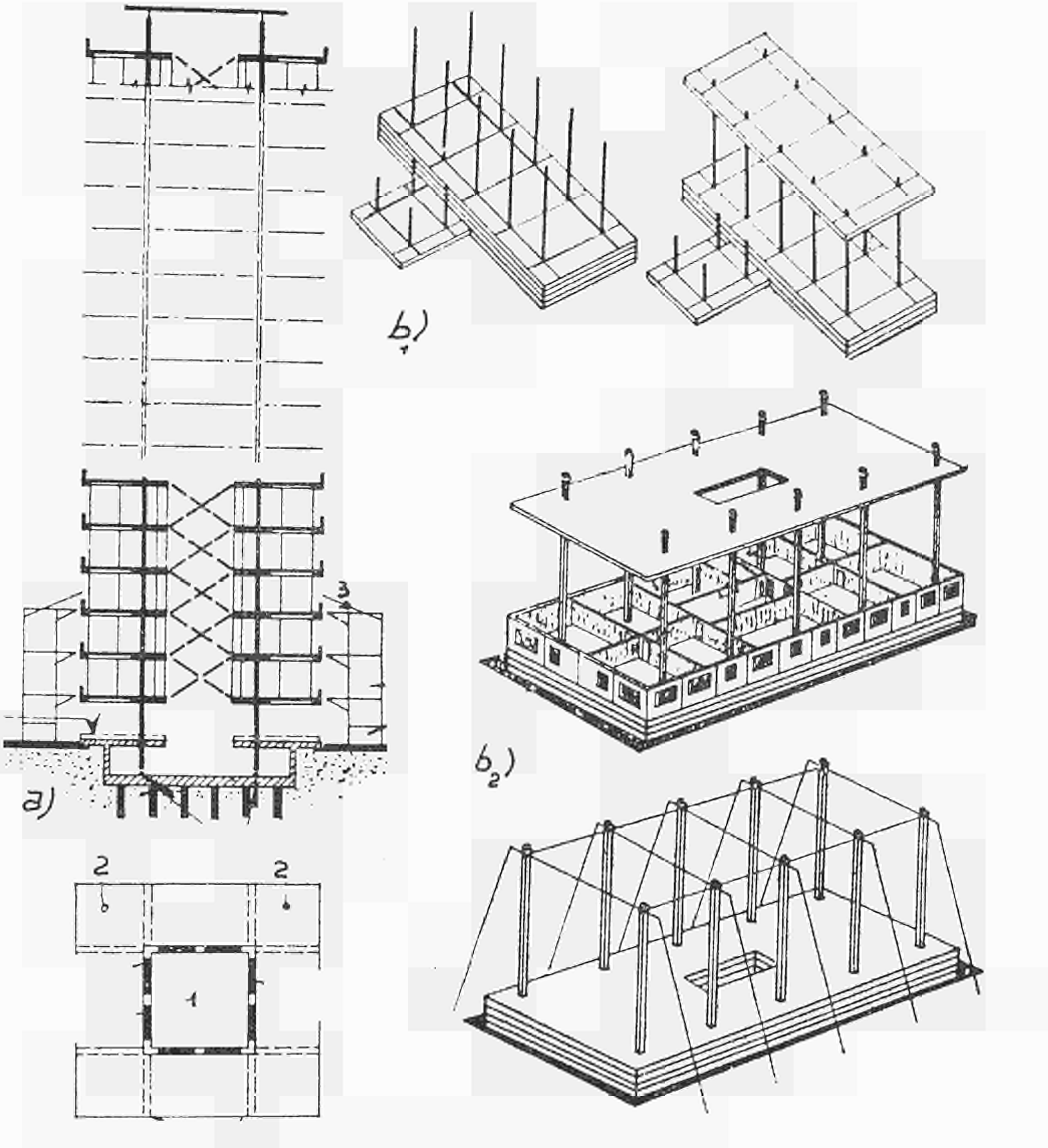


Fig. 16 — Erection of floors by lifting

(a) Tower 1. staircase and lift well

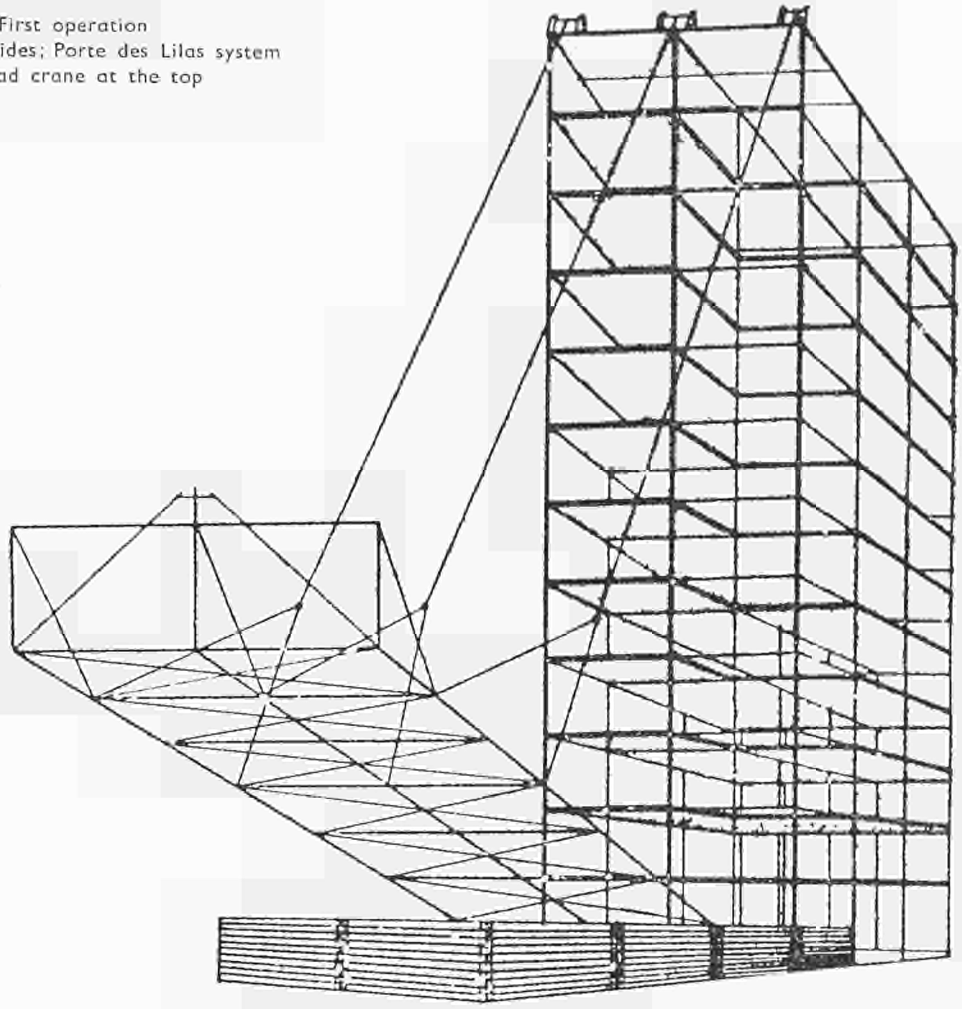
2. cantilever floor

3. light covered scaffolding for completing three floors simultaneously, by the liftslab system.

(b_1) Floors separated by posts

(b_2) Russian system

Fig. 16 c — First operation
— lifting sides; Porte des Lilas system
— Overhead crane at the top



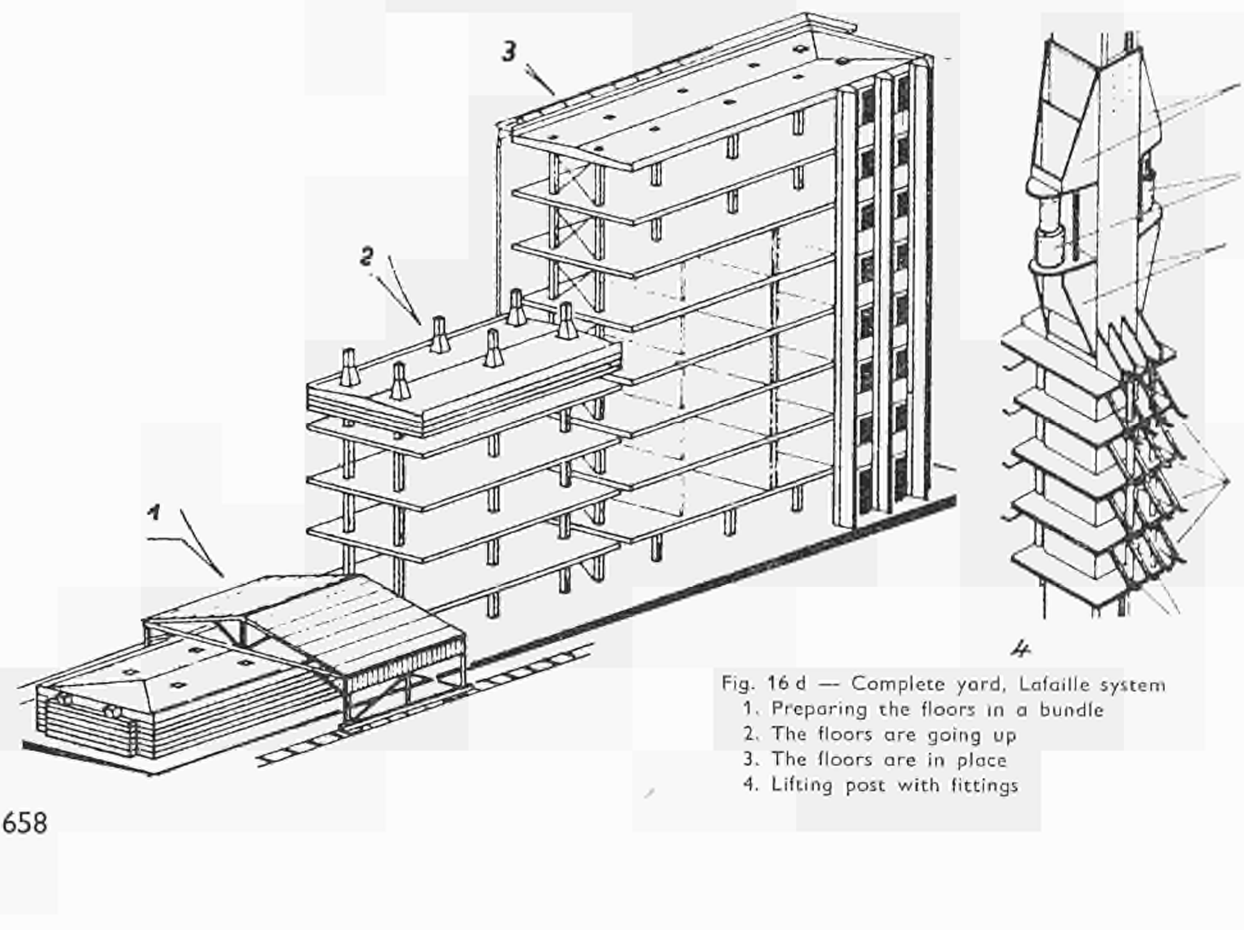


Fig. 16 d — Complete yard, Lafaille system
1. Preparing the floors in a bundle
2. The floors are going up
3. The floors are in place
4. Lifting post with fittings

— With the second system, the building which is much larger laterally, requires many perimetric and intermediate columns so that each floor is divided into so many rectangular, equal shapes, the perimeter of which is defined by the four columns within which they are confined and which serve to lift the corresponding floor components. When the floors are lifted one at a time, it is also possible to attatch to them all the surrounding and internal walls before lifting them. (Fig. 16b).

— With the third system, also included are the entire sides, which in the previous system were built either as the floors went up or completed at the top, using an efficient bracing system during the lifting operation. These instead are built on the ground and lifted complete, by making them rotate round provisional hinges, placed at the foot, often using overhead cranes which are erected at the top and running on the sides already erected. (Fig. 16c).

Erection of skyscrapers

The traditional system remains unchanged: use is still made of various derrick cranes which go up as the building rises. In order to extend the foundation columns so as to form a supporting base for the derricks, use is often made of small self-rising cranes fixed to the posts which rise with them. (Fig. 17).

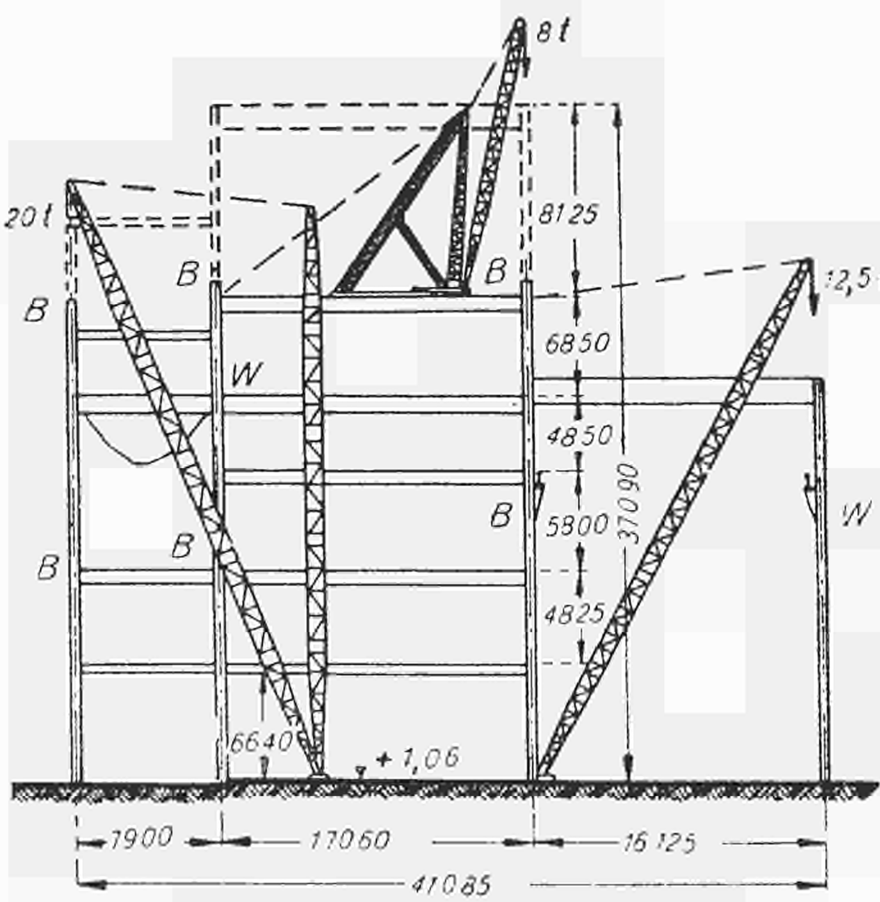


Fig. 17 — Derrick crane for errection work

Erection of high towers

For very high towers of reduced cross-sections the self-lifting rotating jib cranes give very good service. When the towers are considerably wide at the base (halls, churches, temples, etc.), to avoid the use of cranes with an excessive cantilever which would impose excessive bending stresses on the slim structure, it is advantageous to couple the self-lifting cranes to funicular systems as in the example as of Fig. 18, which illustrates the method used by the Author for erecting the spire of the Mole Antonelliana in Turin.

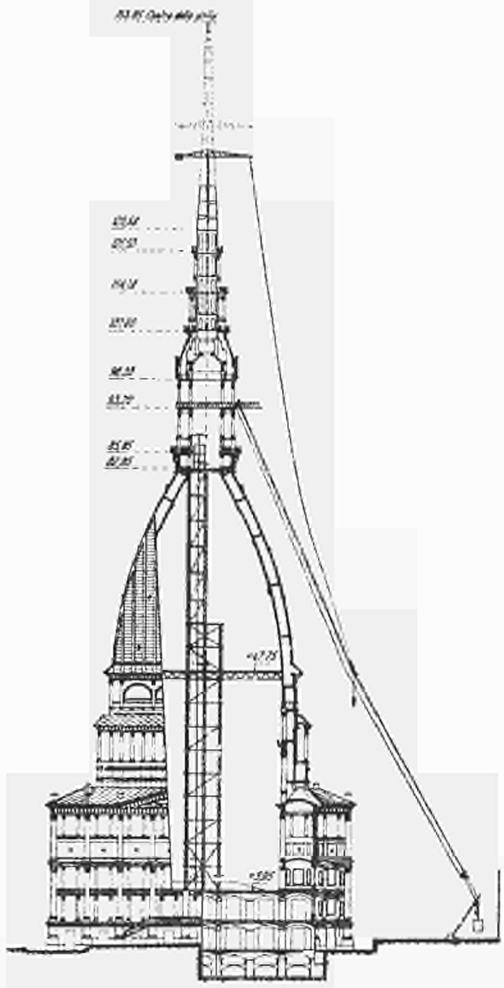


Fig. 18

Erection of large bridges

In this case too, the usable methods are various and numerous in principle. Figure 19 diagrammatically shows those most commonly used.

Special structures.

Steelwork constructions have various other applications, which vary from enormous sluices for large rivers to bridges apu piers in sea and river ports, to flyovers, etc.

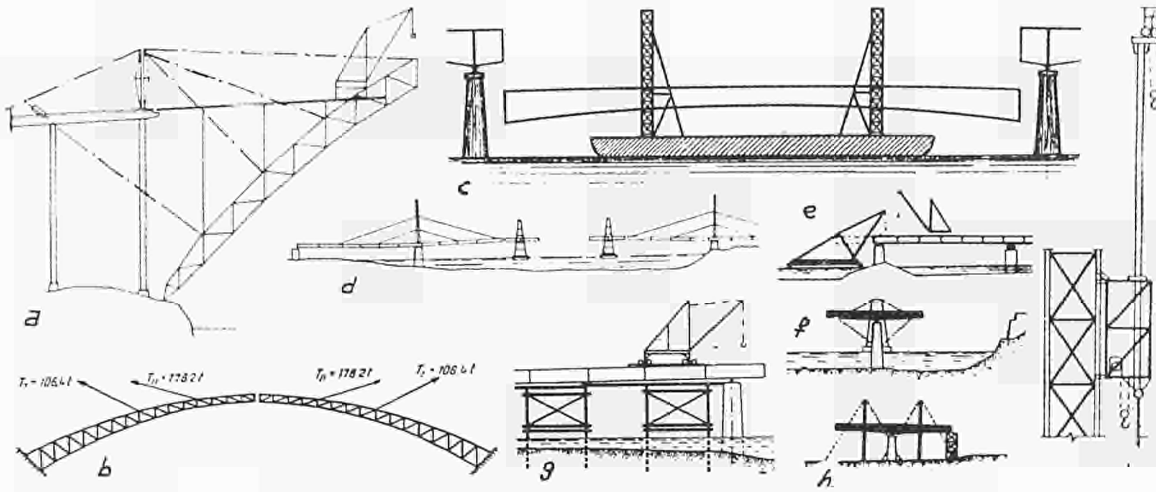


Fig. 19 — The most common erection systems for large bridges
a-b : Erection by cantilevering out arch spans
c-d-e-f : Erection of full structures by floating out on pontoons
g : Erection by cranes moving on provisional piles
h : Erection with 2 derricks
i : Self-rising crane for pile erection

They also actively assist large constructions in reinforced concrete or in concrete, not only with essential equipment such as forms, supports, struts, ect. but also with the centring structures of large bridges which are really like steel embroidery.

In this field too, the inventiveness of the designer can produce solutions capable of effecting considerable savings. I will mention as a good example the system used for the large centring structures for the bridges on the "Motorway of the Sun" which, in order to be re-used for subsequent arches, instead of being dismantled and assembled as is usually done, were removed by lifting them on special trucks, the whole operation taking a single day.

Figure 20 shows one of these structures and figure 21 shows the gear for its transference.

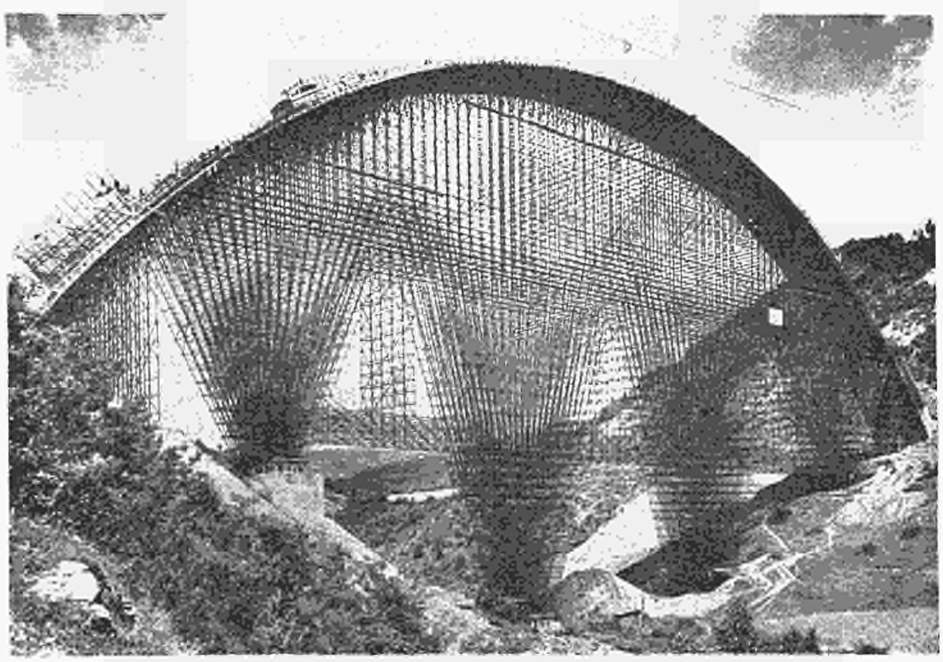


Fig. 20 — Innocenti centring on the Aglio River — Motorway of the Sun
Height, 74 m. — Span of arch, 164 m. — Weight, 850 tons

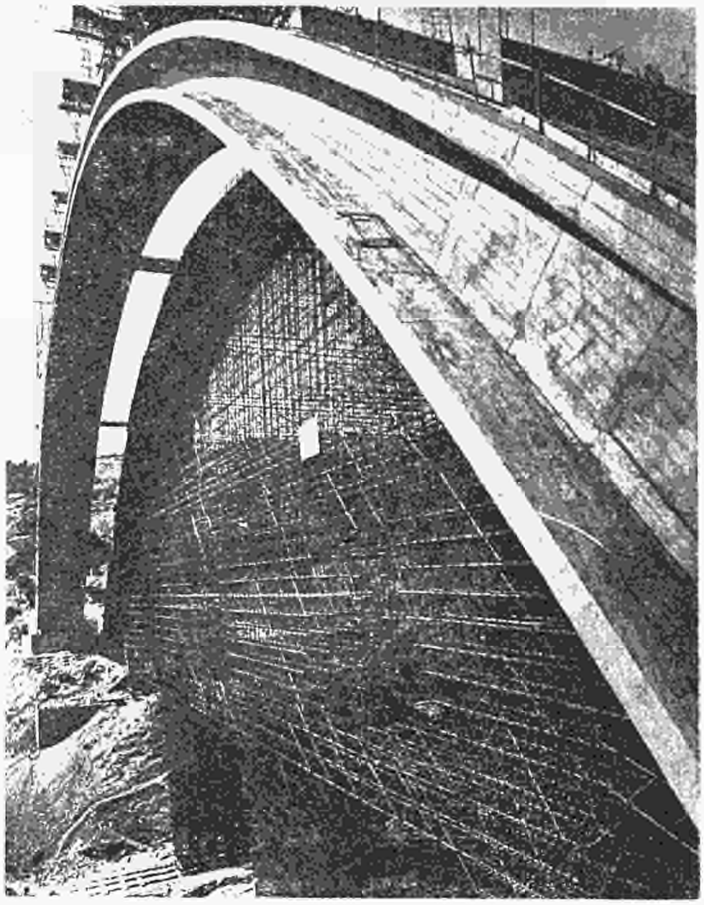


Fig. 21 — Runway for the removal of the centring — Moved by 13 m, in 7 hours

The site

Staff.

The site workers, from the agent to the various specialists, form part, as a rule of the established personnel of the firm, namely the personnel who are well known and can be relied on. Non-skilled labourers can be recruited locally after having carefully examined their capabilities.

The organization scheme of the site must clearly indicate: the personnel required, the diagram indicating the order of command and the respective functions, the duties, responsibility and the limits of the personnel initiative given to each, also how authority is passed in the absence of its holder.

Moreover, it is necessary to indicate clearly the documentary connection between the organs of the site as well as between the site and the general management.

The documentation must be efficient, but reduced to a minimum.

Alongside the organization schedule there must be ready an expenditure budget for the personnel which must agree with the general programme of the works and costs, with the salary and wage rates, and with the individual and collective piece-work rates.

Table of elementary times. The Erection Office (Site Office) will have available the tables of initial times for the essential operations and the price schedules for complete works so that, by applying the current tariffs, it will be possible to know at any moment the hourly and total cost of each operation, which would represent a good check on the working efficiency of the personnel and the organization of the site.

Personal record cards. In respect of each employee, the management will have a personnal record giving brief particulars about each one and his family, the history of his working life, the stages of his career in the Company, his consecutive earnings, his merits and faults, his behaviour at work, in public and in private.

Staff accommodation. Having fixed the expenditure budget for the labour, it is necessary to make arrangements for their accommodation so that they should enjoy certain comforts which lead to a higher efficiency.

According to the extent and the location of the works, the problem may be solved, always under the control of the Company, which should take an interest in the living conditions of their workers, either by making use of local boarding houses or renting local inns or dwelling houses or by installing provisional huts for offices, dormitories, canteens, kitchens, sanitary premises, workshop, garage, warehouses, etc. The normal and estimated cost must be available from the files.

Machinery

The machinery or plant must be carefully selected so as to obtain the maximum profit and usage possible as per operational programme.

Equipment cards. (Fig 22). The available equipment must be classified in the files by means of cards containing all data which can be useful for identifying its main features, namely, dimensions, nominal and actual efficiency, consumption of power, water, compressed air, lubricants, performance already given, depreciation rates, maintenance and repair expenses and their respective dates, and initial capital cost.

The actual cost, which must serve for assessing the influence of the machine on the cost of the work, has nothing to do with the initial capital cost and with the depreciation rates, which are historical informations scarcely proportional to the conditions of the moment.

When the equipment arrives at the site, it must be given a value, equivalent to what the market would be prepared to pay for it, in order to buy it in the condition and at the place where it stands. At the end of the work, the equipment is given the new market value as aforesaid, taking into account the transport charges to be completed. The difference between the initial value (seller's price) and the final value (buyer's price, which is always lower) representing the expense share of the site.

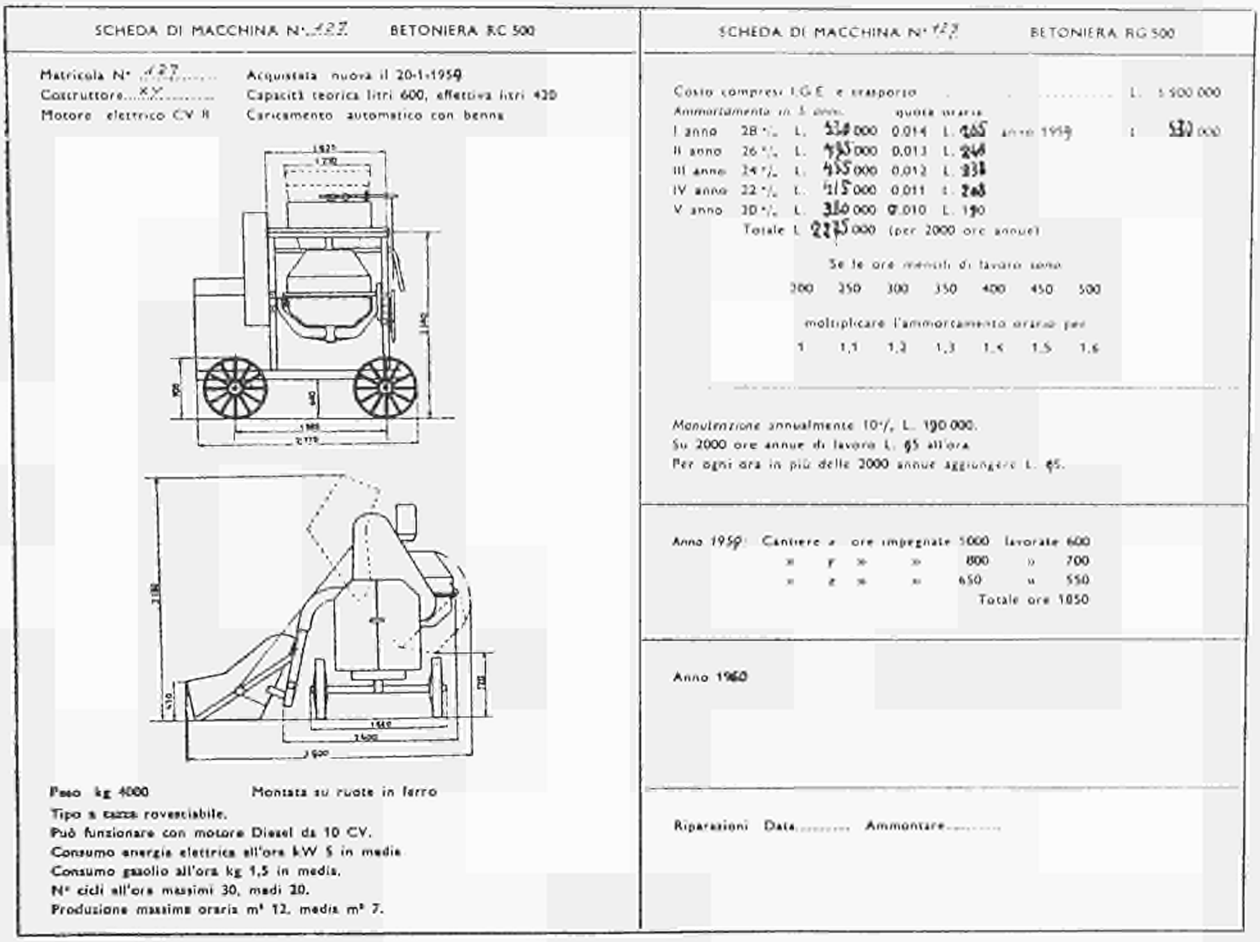
For this computation there are useful, official tables of the costs of equipment, its maintenance and hire which in various countries (U.S.A., France, Federal Germany) are supplied by the respective trade associations.

Service connections

The site needs numerous services which must be planned with in the project and computed in the expenditure budget.

The essential service connections are those in respect of access and provision of the necessary services. The choice of location of the site must be made taking into account the installation and operational cost and the influence that the location may have on erection operations and on the safety of the installations in the case of floods, storms, etc.

Rarely, for example in the case of some sites situated in high inaccessible regions without roads and being works of modest size, has the use of helicopters proved useful for the transport of workers and equipment.



SCHEDA DI MACCHINA N° 187 BETONIERA RC 500

Matricola N° 187
Costruttore XY
Motore elettrico CV 8

Acquistata nuova il 20-1-1959
Capacità teorica litri 600, effettiva litri 420
Caricamento automatico con benna

Peso kg 4000 Montata su ruote in ferro
Tipo a tazza rovesciabile.
Può funzionare con motore Diesel da 10 CV.
Consumo energia elettrica all'ora kW 5 in media.
Consumo gasolio all'ora kg 1,5 in media.
N° cicli all'ora massimi 30, medi 20.
Produzione massima oraria m³ 12, media m³ 7.

SCHEDA DI MACCHINA N° 187 BETONIERA RC 500

Costo compresi I.G.E. e trasporto .... L. 1 500 000

Ammortamento in 5 anni quota oraria

I anno	28 %	L. 530 000	0,014	L. 265	anno 1959	L.	530 000
II anno	26 %	L. 495 000	0,013	L. 248			
III anno	24 %	L. 455 000	0,012	L. 238			
IV anno	22 %	L. 415 000	0,011	L. 208			
V anno	20 %	L. 380 000	0,010	L. 190			

Totale L. 2275 000 (per 2000 ore annue)

Se le ore mensili di lavoro sono

200	250	300	350	400	450	500

moltiplicare l'ammortamento orario per

1	1,1	1,2	1,3	1,4	1,5	1,6

Manutenzione annualmente 10 % L. 190 000.
Su 2000 ore annue di lavoro L. 95 all'ora
Per ogni ora in più delle 2000 annue aggiungere L. 95.

Anno 1959: Cantiere	x	ore impegnate	1000	lavorate	600
»	y	» »	800	»	700
»	z	» »	650	»	550
				Totale ore	1850

Anno 1960

Riparazioni Data......... Ammontare.........

Fig. 22 — Machine record card

Machine card No. 187 — Cement mixer RC 500
Series No. 187 — Purchased new 20.1.1959
Maker : XY — Nominal capacity 600 litres, actual 420 litres
Electric Motor : 8 HP — Automatic loading by grab

Weight, 4,000 kg
Mounted on steel wheels
Type : reversible bucket
Can work with 10 H.P. Diesel engine
Average gas oil consumption per hour : 1.5 kg
Max. No of cycles per hour : 30 – average : 20
Max. production per hour : 12 cu.mt., average 7 cu.mt.
Cost, including IGE Tax and transport : 1,500,000 Lire
Depreciation in 5 years ---- per hour

1st Year	28%	L. 530,000	0,014	L. 265	year 1959
2nd »	26%	L. 495,000	0,013	L. 248	
3rd »	24%	L. 455,000	0,012	L. 238	
4th »	22%	L. 415,000	0,011	L. 208	
5th »	20%	L. 380,000	0,010	L. 190	

Total 2,275,000 (per 2,000 yearly hours)

If the monthly hours of work are

200	250	300	350	400	450	500

multiply the hourly depreciation by

1	1.1	1.2	1.3	1.4	1.5	1.6

Annual upkeep cost 10% = L. 190,000
For 2,000 working hours p.a. = L. 95 per hour
For every hour in excess of 2000, add L. 95

Year 1959. Site	x	Hours engaged	1000	worked	600
»	y	» »	800	»	700
»	z	» »	650	»	550
				Total hours........	1,850

Year 1960
Repairs : Date .... — Amount ....

Road and rail connections. These are necessary to connect the site with the rest of the country whence goods and labour proceed. Consequently, road and sometimes rail, connections are required, also in order to connect the builder's yard to the site or sites, which must be done with maximum facility and minimum expenses.

Power. The planning of the yard includes the power schedule in respect of the various items of equipment (installed power) and the total maximum power required for operation (generally lower than the installed value).

In general, it is preferable to connect up with existing power lines. If this is not possible, a generating set should be used arranging for each machine to be driven by its own electric motor.

Only in rare cases, for instance when all operations are able to function on compressed air only would it be advantageous to use an internal combustion engine set (always supplied with a spare).

Drinking water and industrial water. If possible, it is advisable to connect up to the existing mains. If not, arrangements must be made for conditioning the water, (obtained through the available means), very carefully in order to make it drinkable.

Compressed air. When necessary, this is obtained either by self-propelling sets or, in the case of important works, by stationary equipment.

All essential services must have an alternative in reserve. and for each of them, a calculation must be made of the cost of installation and operation, so as to obtain the cost per unit supplied (kWh, cubic metre of water, cu.metre of compressed air, cost of workers per man-day, etc.)

Cost of the yard

The sum of the installation costs mentioned above gives the total installation cost of the yard. From this, taking into account the costs relating to the staff in charge of the various services (cleaning, supervision, motor vehicles, etc.), it is possible to obtain the operating cost per day and for the total time planned for the erection.

Cost of erection

In general, the cost of erection is given by the sum of the various items of Table II which are obtained from the actual work carried out and expressed in hours of a skilled worker, thus avoiding the use of a measuring unit so variable as the money values.

Execution of the plan

The execution must be followed up day by day, comparing the estimates with the actual results as regards times, progress, disbursements and costs.

It will then be possible to trace any anomalies, such as delays, increased costs and climatic difficulties. To diagnose and correct them, reducing all errors to a minimum.

For checking the productivity of machinery and labour, the fundamental factors of the organized production are very useful.

Table II

Example of the cost factors of an erection job.
Cost factors per kg. of structure

	Costs (hours of skilled worker)
Erection management	0.0015
Skilled labour	0.0200
Labour for preparing yard and handling plant	0.0014
Overheads for the erection	0.018
Labourers for transport and assistance	0.021
Depreciation of handling plant	0.0028
Consumable materials	0.0017
Medical care, first aid, workers' insurance	0.0030
Land occupation and third party claims	0.0010
Overheads of Head Office for technical office and planning the erection	0.0030
Overheads of Head Office for services	0.0010
Total hours of skilled worker	0.0744
Allowance on above cost for contingencies, 15%	0.0112
Pay office and staff	0.0050
Transport from workshop to site	0.0150
Recovery and return of handling plant and re-arrangement of land	0.0024
Total cost of erection and transport	0.1080
This was a difficult erection, for which the cost of the materials, always on the basis of hours of skilled worker, was	0.10
The cost of the work in the workshop, including overheads and all other charges, in hours of skilled worker was	0.12
Total cost in workshop	0.22
The cost of the skilled worker, excluding overheads and charges included in the previous computations was 600 Lire per hour.	
It follows that the costs as determined were :	
— Materials per kg.	60 Lire
— work and overheads in workshop	72 Lire
Total for materials in workshop	132 Lire
— erection, approx., rounded off	45 Lire
— contingencies, transport, pay, return of plant	20 Lire
Total cost of bridge erected. Per kg. :	197 Lire
September, 1964	

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Gerrit DERKZEN

Difficulties Encountered in Steel Construction in the Netherlands

(Original text: Dutch)

Speaking as a consultant to large and medium-sized firms, I regret to say that it is still no easy matter in Holland to move away from the traditional way of doing things in building. This is very often the fault of the client, who knows what he wants his new premises or extension to look like but does not know anything else about it. It would be preferable for there to be consultation, and closer examination with experts of such matters as routing, layout, location and transport before building operations start. Unfortunately, the client has very often taken such a time to make up his mind to build at all, that he jibs at postponing the start for another six months or more.

Now this is just about the worst background to building in steel that can possibly be imagined. A concrete structure can be altered at any time up to the last minute, but a steel structure needs planning. Nevertheless, to my mind all this constitutes an argument for more building in steel. After all, the small or medium-sized firm will need to concern itself more with preparation if it is to exist.

Another continuing obstacle to building in steel is that during assembly it is often necessary to work amid the traditional building trades. I should like to make one or two points in this connection. Firstly, the supplier of the steel structure is often represented on the building team only whilst the frame is being erected. Secondly, though the main contractor is often willing enough, he is usually not accustomed to building with prefabricated components, and is unlikely to deal competently with the work of assembly and erection. Thirdly, there is the difference in tolerances between steel and, for instance, concrete components: it is impossible to over-emphasize the importance of an exchange of drawings between the supplier of the steel frame and the manufacturer of the roof and façade. Fourthly, if the elements surrounding the steel frame is workshop-made, it is desirable, even if these other components are to be of concrete, that the supplier of the steel structure should act as main contractor, being responsible for the roof and façade construction. This arrangement would be found to pay off during assembly and erection.

Paul BOUÉ

Planning and Co-operation in Steel Construction

(Original text: German)

You have just heard how we in Germany have been thinking along the same lines, and are on the way to organizing co-ordination in the way we think best suited to German conditions. But of course this is but a start.

The point was, in industrial building we have to deal with three sets of client's representatives, as well as with the client himself. In the first case, the client has a building department responsible in its own right for co-ordination (as for instance with big chemical works and the like). Generally, the constructional steelwork engineer is among those associated with the work from an early stage, and so can assert his interests, in order to ensure streamlining right through to the building operations themselves. In the second case, the architect takes on the job of co-ordination. Here you sometimes get excellent co-ordination, but more often than not, unfortunately, you get utterly faulty planning. Usually it is even worse still when the client acts entirely on his own.

To my mind, this is really the concern of the constructional steelwork company, which should itself take on the whole thing and carry it to completion. It should be in general charge and as main contractors, should be responsible for overall planning and co-ordination. Take the case of a steel frame structure: I am thinking of one particular four-storey office block (I am afraid I have no illustrations) for which the constructional steelwork company did all the planning and co-ordination, with the architect responsible only for the design. Arrangements were simple, since we installed not only the frame but also the steel ceilings, the internal fittings and the steel roof; the wall components were mostly steel panels of storey height with built-in windows. The partitions, and with them of course the service installations were constructed by us, sometimes with plumbers, electricians and so on acting as sub-contractors.

Then again consider for example the construction of shed-type buildings for storage and production purposes. Here we have a standard programme on the unit system, adaptable to different circumstances; we supply not only the steel structure but also roofs and walls, continuous window-panels and sky-lights, doors, and last but not least the crane gear. In some cases we have even extended our planning to include heating and lighting.

The implications of what I have been saying are twofold. I will sum them up once more. First, architectural training should be focused more on comprehensive planning, and away from the "adjust-as-you-go-along" attitude, and stress the potentialities of steel construction. Secondly, constructional steelwork firms should be responsible for the whole connected operation of planning and delivery right up to the assembly/erection stage. I feel this could help to increase productivity and enhance the long-term competitive capacity of steel construction.

Claude DUVAL

Productivity in the Assembly of Steel Framework

(Original text: French)

For ten years our company has had a special department devoted to the increase of productivity in steel construction work and boiler making.

Applying the Cartesian principle "divide problems into as many parts as necessary to resolve them" we started each field with the easiest thing so as to obtain incontestable results. The workshop was therefore the subject of our first investigations.

Study of work shifts, design of new machines or new plant assembly, of installation and maintenance, etc.

Management and its orientation were dealt with.

Sixty "guides" arising from many discussions with management and staff have marked these ten years.

The results are significant. Most shops have doubled output without increasing staff.

The systematic study of on-site assembly began only four years ago, since this problem is much more complex. Even now we have only probed the average and repetitive sites in respect of forms and measurements.

We considered that on one hand we needed an accurate, tried and tested method to deal with the assembly of large specialized units, and on the other that we must first spend time on normal sites which made up two-thirds of the output of the group which had entrusted us with the investigation.

Introduction

A site

— a plot of ground owned by the customer, on which a foreman heads the team which assembles the constituent units of a given structure. The units are delivered before or during assembly.

Operationally:

— a succession of occupations*, that is a sequence of periods devoted to repeated or different operations.

Observation of each site individually can lead one to believe that each is a special case.

Proceeding with the investigations, one can state that all basic occupations** on each site are very similar. From this one can deduce precepts for most sites provided that one can analyse in detail all occupations or operations.

Memo-film

Assembly work in steel construction is not an easy thing to understand in terms of traditional work-study methods. The parameter of the working space for a group of workers on the same assembly must be taken into account. One cannot observe all the details of more than two occupations within ones field of vision if the operations are of short duration.

It was therefore necessary to find another study method. Our choice fell on a ciné-process, the memo-film.

Let us give a simple explanation. Watching an operation one can ascertain that the workers' actions and the movements of equipment are not instantaneous. They have a certain duration. If, instead of observing work continuously, it is observed at intervals, to have a precise idea of the duration of each operation, one needs to have at least one observation at the start and one at the finish of each operation. If there are many simultaneous operations which interconnect and disperse variously during the period, the observations must

* By occupation we mean the way the time is utilized.
** By basic occupation we mean the various hoisting, unloading and hooking up actions.

be arranged so that the start and finish of each operation is clearly defined in order to find the durations accurately.

A method which permits the ascertaining and recording of duration and period by means of regular, short interval all-round observations can be substituted by an adequate continuous observation process (applying the theory of instantaneous observation).

The memo-film process is one which uses this method.

It consists of taking photographs at regular intervals of the operation to be observed, then the projection of these photographs. Each one shows the state of the operation after each equal exposure interval. Interpretation of these photographs shows the advantage of seeing the work at the same time as its duration is calculated.

The memo-film studies conducted in the U.S.A., Sweden and France have neccessarily taken place on work near to a source of electric current to drive the electric motor which activates the automatic shutter release. However, steel construction sites do not always have electricity within reach of the camera.

So we had to consider a means of photography with independent shutter operation. The apparatus used comprised a single-shot camera, an electronic time-elapse movement (an apparatus which switches the electric current on and off to a set pattern) and a system of relays, activating the shutter release. The apparatus worked on a battery and proved to be quite satisfactory.

Having made the apparatus, it was now necessary to experiment and to perfect the method of working, which comprised analysis of the part of the structure already erected, deciding the shot pattern, selection of angles, synchronisation of film and observation notes, summarizing the film, and exploitation of the results.

Part I

Investigation of time spent

Chief categories of time spent on site

(1) The time for the actual work of assembly and connected operations (occupation, unloading, storing, approach, preparation, true assembly operations, adjustment, packing up, break time) vary from one site to another from 36-45% of the paid time.

(2) Waiting time during assembly varied from 22-37% of paid time.

(3) Time devoted to various other occupations varied from 17-44%.

It can be seen that under the best conditions with current methods and usage, 39% of the time is not spent on necessary work. It is a known fact that analysis has enabled the reasons for this to be studied, as well as the "effective working time".

Types of occupation on the sites and their relative importance

(a) The fundamental, necessary occupations are well known:

— determining the assembly operation schedule
— ground level assembly (2-8.5% of the time)
— actual hoisting (1.25-2%)
— positioning and bolting (8-12%)

= 12-22.5%

(b) Auxiliary operations comprise:

— bringing in units for assembly	(6.5-8%)
— locating auxiliary units, supplementary material and equipment	(2.25-3.25%)
— preparing hoisting equipment	(1-1.25%)
— shifting position	(4.50-6.25%)
— hooking up, coupling (slinging) and uncoupling	(0.25-0.50%)
— levelling, plumb-lining and adjustment	(2.7%)

= 10-19%

(c) General occupations take up 9.9-12.6% of the time and comprise

— unloading lorries
— installation and packing up
— rest time etc.

(d) Finally various occupations. 17-44% are taken up by

— break-downs
— location
— bad weather
— administrative formalities.

Examples of distribution of time by occupation on four sites
a) Total time

Example	Total time for site	Actual assembly 1	General Occupations: Unloading lorries 2	Installation and packing up 3	Adjustment 4	scaling 5	Foreman working hours* 6	Theoretical total (1-6)	Various: Accidents	Various	Tonnage	H/T ratio: Actual	Theoretical total (1-6)
1. SHED no breakdowns concrete floor, motor driven equipment	1526 h	826 h	66 h	28.4 h	30 h	70 h	176 h (a)	1196.4 h	329.6 h		125 tons	122	9.5
			4·6%	1·9%	2%	4·6%							
	100%	54%	13·1%				11·6%	78%	22%				
2. SHED motor driven equipment with portable crane	841 h	247 h	40.5 h	23.15 h	50 h	49 h	62 h (b)	472 h	77.5 h 9.3%	292.4 h 34·6%	40 tons	21.4	11.8
			4·8%	2·8%	5·9%	5·9%							
	100%	29·4%	19·4%				7·3%	56·1%	43·9%				
3. PH105 with Dodge (not used)	320 h	113 h	already supplied	17.5 h	14.7 h	16 h	21.5 h (c)	181.1 h	31.6 h	106.7 h	15.8 tons	20	11
				5·6%	4·6%	5%			9·9%	33·1%			
	100%	35·1%	15·2%				6·7%	57%	43%				
4. PH105 solid framework	576 h	274 h	25 h	35 h	38 h	22 h	72 h**	466 h	99 h	11 h	45.1 tons	12.5	10.2
			4·3%	6·1%	6·6%	3·8%			17·2%	1·9%			
	100%	47·6%	20·8%				12·5%	80·9%	19·1%				

* When foreman did not work as crew member
** Foreman and cranedriver when latter did not work but had to be on hand.

(a) Foreman worked 36 hours (17% of his time) as crew member.
(b) Foreman worked 88 hours (58·8%) as crew member.
(c) Foreman worked 18,5 hours (46%) as crew member.

b) Basic operation in relation to total time for sites in examples 1, 2 and 3:

	Site total		100%	100%	100%
	Total time for assembly	≠	54	29·4	35
	Waiting time	≠	18	7·5	10·8
	Working time	≠	36	21·9	24·2
Break down of working time	Assembly	≠	7	0·5	1·9
	Hoisting	≠	1·7	1·6	1·3
	Positioning/Bolting	≠	10	8·2	8·6
	Bringing in	≠	5·3	2·3	3·4
	Tooling up	≠	5·5	2·8	2.8
	Preparing to hoist	≠	1	1	1·2
	Uncoupling	≠	0·4	0·1	0·1
	Travelling on ground	≠	3·3	3·8	3·1
	Travelling on building	≠	1·8	1·6	1·6

Example 1 was a site made the subject of experimentation and used for training the foreman.

Example 4 was a site prepared on the principles given in the following pages.

Examples 2 and 3 are sites reflecting present trends.

Part II

Winning back lost time

Study of component parts of the site and their importance in productivity

(1) Component parts and their present state.

Ground. An important point which affects the various occupations. Knowledge of topography, preparing access for equipment, accessibility to stores.

Present state. Poor knowledge, non-existent or improvised preparation, arbitrary selection of storage point.

The building. Weight, dimensions, types of attachment, location.

Shop-site liaison. Delivery schedule, type of load. Non-existent time-table or schedule, knowledge of make-up of loads poor.

Means. Inventory of materials, selection of coupling method, workers' equipment, selection of plant, battery run, adaption to ground, safety during hoisting, manageability. Initiative is greatest in choice of material, workers' equipment basic, materials are used as they arrive. Equipment very often does not fully conform to needs and is not suitable for all types of ground.

Foreman. Information on methods, time, organization, distribution of work, coordination of several crews, safety control, supervision of operations, control of work estimates, of execution and of uniformity of units.

At present the most efficient methods are not usually known, nor is research into operational methods on the half-built structure, disregard of the duration of each operation, laxity over safety, units not arriving in the correct order, and haphazard arrangement of occupation. Ignorance of delivery schedules, and only rarely planning for more than one day ahead.

(2) The results of the present situation can be summarized by an example.

The waiting time comprised:

(in %)

Total waiting time	Site A	Site B
Ground	1.10	2.44
Building breakdown	11.00	5.00
Method	30.65	42.85
Foreman	26.00	33.40
Equipment	13.00	7.15
Plant	10.20	1.77
Bad weather	2.70	2.44
Various (1)	5.45	4.75
Total	100	100

(1) We should remember that waiting time during work can vary from 33-51% of the time devoted to it.

Winning back lost time

In the short term from the first investigations one action was obvious:

— make site preparation at service level
— education of site foreman

In the long term action on

— making an index of time spent
— cooperation of the assembly study office on the planning of buildings.

Immediate action:

I Site preparation

There is of course some risk in the operations on a site. But it is essential we know how far this risk extends and to which aspects of the methods it applies. In fact there is plainly a lack of preparation in three aspects

(1) Research in operational methods which can be done beforehand

(2) Research on storage location. One can use *a priori* determination within quite acceptable limits.

(3) Research on remedies for hazards. Most can be planned after a number of analyses.

The required conditions for site preparation are

(1) those in charge must be convinced that improvisation is expensive; that in any case there is preparation even if its presence is not at once apparent;

(2) the site foreman must be trained for his task. Preparation will be ineffective if the foreman does not know how to benefit from it;

(3) the teams must be provided with equipment determined beforehand. It is no good preparing if the equipment to carry out the work planned is not to hand;

(4) site preparation in accordance with the overall plan must be made, and the post of site preparer created;

(5) keep to essentials. Do not prepare with insufficient or doubtful data.

Preparation method

(a) Study of basic data:

— promises made to clients and desiderata;
— structure of the building;
— ground and environment: Only investigation of the site can provide all the necessary information. To be of use it must be conducted with a check list.

(b) Selection of overall method *i.e.* arrangement of hoisting operations and working out crew occupations.

At present the difficulty is in not having prior knowledge of operation durations but for the overall method one can make do with an approximation of the order of 20% without the risk of great error.

As a progressive time and motion index is made up this difficulty will be shaded out.

(c) *Planning:*

— storing and delivery plan;
— list of equipment needed for the work given in detail;
— details for equipping the crew;
— alternative methods.

(d) Control:

— planning of assembly and plant;
— ratio of foreman.

To facilitate the use of this method a memorandum on the preparation of sites and a practical example of current site preparation has been published

2. Training of site foreman

For many workers the site foreman is a more advanced crew member, an executive crew leader

— the site foreman actually works as crew man for at least half the time;

— the training of crew and replacements is something which does not occur to them;

— the function of control which is an essential quality for the executive leader is almost abandoned;

— the foremen spend their time thinking up operational methods while the crew waits, and at each site the foreman repeats the same errors.

Here is an example of a table of occupations of 2 site foremen.

Foreman's work	Foreman A	Foreman B
Obtaining information		
— reading plans	0·25%	5·6 %
— consultation with the engineer/architect	0·56%	1·2 %
Total	0·81%	6·8 %
Coordination and regulation		
— work distribution	0·25%	2·8 %
— indication of equipment and method	0 %	0·5 %
— directing plant movement	0 %	11 %
— coordinating crew operations	0·25%	11 %
— discipline	0 %	1·8 %
Total	0·5 %	27·3 %
Training		
— explaining the work	0 %	2·8 %
— practical demonstration and helping crew	0 %	1·8 %
— setting example in safety	0 %	0 %
Total	0 %	4·6 %
Control		
— examining the premises	0·56%	1·7 %
— examining equipment and losses	0·28%	0·93%
— examining deficiencies and amount of scrap	0·7 %	2·8 %
— daily examination of work progress	3·6 %	18 %
Total	5·14%	23·45%
Overall total	7 %	63 %
Work as crew member	93 %	37 %
	100 %	100 %

There can be no doubt that site foremen need to be trained. The only problem is to know first of all whether to train those who are already practicing foremen, and how and when, or to call on other foremen. Both arguments have been retained.

(a) To acquaint the site foremen with the essentials of their duty a memo for site foremen was drawn up, which comprised the rôle of the site foreman—he is a leader of men, a cadre, an assembly technician, his activities : information, planning, organization, coordination, control, discipline, training, safety, representation, assembly, the method he uses in running the site:
  - — when the site is announced;
  - — on starting;
  - — arriving at the location;
  - — on the site;
  - — on completion day.

(b) Training sessions based on the systematic study of sites analysed on memo-film were organized. They are now being followed up by regular on-the-spot training for future foremen who take part in these sessions within the company.

These training sessions deal with the practical aspects of site life and initiation into the methods of simplifying work, T.W.I. and the art of organizing one's time as executive foreman.

It is also necessary to mention points like reading and interpreting plans, costing, levelling and all the technical elements which facilitate the work of an erector.

Means:

The means at the crew's disposal on the site very largely condition the efficiency of its work.

Of these, plant has the most effect on productivity.

The contractor has a choice between:

— no equipment

— chance equipment

— expensive equipment

Now all the analyses made on sites show that on one hand the use of mobile plant for off-loading and hoisting operations is unavoidable for acceptable assembly times and on the other the use of powerful cranes is not always profitable on modern sites, the crane being used in effect for 2-3 hours and costing the equivalent of 2-5 workers.

The object is not to find plant suitable for all sites. Various types must be studied.

Our first investigation was of plant handling units up to 1 1/2 tons with a hoisting range of 12 metres.

This type of site represents approx. 75% of cases for the majority of companies in the group for whom we made this study.

Taking into account modern assembly methods* a mobile crane is best adapted and the least difficult to use.

In collaboration with a plant designer, we studied the adaptation after modification of a telescopic mast mounted on a forestry tractor.

A prototype is now in service to the great satisfaction of site foremen.

In the future an economical crane for sites must be investigated. Close cooperation with manufacturers of this equipment is necessary and probably financing of studies and prototypes.

Conclusion

While the on-site assembly of steel frameworks looks like being more expensive than their fabrication in the shop, the serious detailed studies made in this field are only a starting point.

They do however interest hundreds of builders and assembly men.

The first studies indicated the possibility of considerable increases in productivity (20-50%).

It would probably be desirable to correlate the information and finance a greater, international programme to analyse a large number of differing sites.

These analyses would permit the effecting of

— valuable data on working time
— a method of preparation and control
— a programme and means of training site surveyors and site foreman.

The orientation and coordination of these studies by the E.C.S.C. would come within the scope of its preoccupation, at the same time enabling steel to become more and more competitive.

* Today, assembly takes place in a sequence of hoisting operations of vertical elements (columns, trusses, girders for the most part) which are joined to horizontal members (windbracing purlins) hoisted later. This is why the low power mobile mast is well adapted. If large units assembled at ground level are envisaged, a powerful crane must be resorted to.

Example of a reconstructed time sheet from which a time schedule can be made.

Hoisting portal frames

Weight hoisted each time 1560 Kg			PF a	PF b	PF c	PF d	Av	%
Total time (in minutes)			125	87	96	159	120	100
Waiting time			16	2	3	9	8	6·7
Working time			109	85	93	150	112	93·3
% occupied			87·3	97·8	96·8	94·3	93·3	
Occupations	Basic occupations	Hoisting	20	6	6	18	13	10·8
		Positioning	20	33	39	60	38	31·6
	Connected occupations	Locating Bolts and tools	—	—	3	—	1	0·8
		Preparing for hoisting	12	12	12	18	14	11·7
		Uncoupling	6	4	2	4	4	3·3
		Connected Operations	40	24	29	46	35	29·2
	Moving	On building	8	6	2	2	5	4·2
		On ground	3	1	—	2	2	1·7
Duration of assembly			36	32	31	52	38	

Plant	Total time		29	31	44	37	100
	Working time		27	19	27	27	73
	Waiting time		2	12	17	10	27

Example of a time summary for a whole site

Reconstruction of hoisting

kg, hours, minutes		Number			Weight		Duration*		Crewtime*		H/T	Plant time	
		Assembly unit	Units: Total	Units: Assembled at one time	Average per ass. unit	Total kg.	Average per ass. unit	Total	Average per ass. unit	Round Total (hours)		Average per ass. unit	Total
Portal frames		14	56	4	1560	21.840	3 h 10	44 h 20	7 h 10	100 h	4 h 30	52 mn	12 h 08
Sills (brace, tie)		26	26	1	145	3.455	1 h	26 h	1 h 30	38 h 30	11 h	10 mn	4 h 20
Purlins	Ridge purlin	13	13	1	65	845	8 mn	1 h 40	17 mn	3 h 40	9 h	8 mn	1 h 40
	sloping	26	195	78	486	12.625	10 mn	17 h 25	1 h 25	37 h	3 h	6 mn	2 h 40
Wind bracing	Vertical	2	4	2	85	310	50 mn	1 h 40	1 h 40	3 h 20	10 h	—	—
	sloping	12	24	2	42·5	510	50 mn	10 h	1 h 40	20 h	40 h	—	—
Cladding	Columns	6	6	1	180	1.080	13 mn	1 h 20	25 mn	2 h 30	2 h 30	12 mn	1 h 12
	Ties	72	72	1	48	3.456	16 mn	19 h 12	31 mn	37 h	10 h 30	—	20 mn
Minor assemblies		—	—	—	—	875	—	—	—	32 h 20	37 h	—	—
Total					≠ 45.100				274 h 20		6 h 10	22 h 40	

* Off-loading and assembly

Site H/T 12 hours 48 mins.

Franco BIANCHI DI CASTELBIANCO

Suggestions for Furthering the Cause of Steel Construction

(Original text: French)

Before you can take on the organizing of the assembly and erection of a steel structure, and particularly of a steel frame building, you have to be asked to! As we know, this does not depend on the structural steel fabricator, since in many cases he only supplies comparatively minor portions of the structure: the figure of 15% has been mentioned, or at best, I think, 25-30%, not more. We are usually in the hands of the architect or the client, whose main concern is the overall cost of the project. It is not easy for the fabricator to show that a reduction in the cost of the frame of the building is bound to bring down the cost of the whole. In Italy, we are facing stiff competition from reinforced concrete; if we are to make a push to promote steel construction, we shall have to act as general contractors, and to show clients that the whole structure will cost them less if they order everything from the structural steel work company. We shall then have to see whether, in carrying out the project, we can confine ourselves to supplying the steel frame work and rely on the co-operation of other firms and trades, or whether we shall have to take on a bigger responsibility and see to the assembly and erection and even the subsidiary work ourselves, *i.e.* prefabricate some components other than just the steel framing itself. Of course we can study the matter beforehand, but not until we have managed to convince the client that steel construction is what he needs shall we be really able to make headway.

One can distinguish various aspects here. One has the very large buildings for which steel framework is pretty well a „must"—the skyscrapers. However, these are not very common, and do not think the present trend is towards a very much higher proportion of them. Even so, we could still make a drive to increase our efficiency in assembly and erection and in site organization.

Then, one has the one- or two-storey industrial building. Here steel construction is often rather at a disadvantage, because it is not noticeably cheaper than its competitors: building in brick and other non-metallic materials is highly economic nowadays, and has to be borne in mind. Nevertheless, this field has immense possibilities: the steel fabricator can do practically everything—produce not only the skeleton but also the roof, walls and many other components —without getting too far outside his usual range.

Between these two extremes, one has office and residential building, where there is both mass and individual construction. I do not think we should devote much attention to the individual side: the architects can at present follow their personal preferences in design, and steel building is not highly regarded. What we should concentrate on is the mass construction field—big blocks of low-rent flats or of offices, where building in steel really can compete very effectively with the traditional methods. And there, perhaps, we may be able to break the vicious circle in which we are at present caught—no organization because no orders, and no orders because no organization. I think what Prof. Triebel has told us about the efforts they are making in Germany in this regard should be something of an example to everyone. But I also think that if building in steel is really to make progress as against other modes of construction it has got to be extremely flexible: that is to say, methods of construction must be devised which will allow the greatest scope for the other installations and parts. For example, if a steel construction has no internal supports, only external ones, and all the storeys are entirely free, it will be easy to arrange and fit them out in whatever manner the architects, big companies or Government departments have in mind, that is, to split them into larger or smaller sub-units. If in steel building we adhere too rigidly to set formulas, it will be more difficult to satisfy these different types of client.

Konrad GATZ

The Great Majority of Architects do not have much to do with Structural Steelwork in Practice

(Original text: German)

No major expansion of the use of steel in building is really possible unless there is a fundamental change in the attitude of the architectural profession as a whole. To my mind, at this Congress, which was specifically organized to find ways and means of stepping up sales of steel for building, we have heard too little from the architects: the proceedings have been too much confined to the steel men, who, after all, have very little say in the everyday decisions as to the building systems to be used, except in the case of purely engineering projects. Those decisions are for the most part a matter for architects (and their clients, of course).

With comparatively few exceptions, architects still tend to look at steel with a vague feeling that so far as load-bearing superstructures are concerned it is only, or at any rate mainly, suitable for purely engineering jobs or for certain types of large structures, which are also very often a matter for specialists. In Europe however probably more than 80% of architects deal exclusively throughout their careers with small and medium-sized structures: only the remaining 20% ever have to do with larger projects as well, and only a few with actual engineering works.

If, then, the great majority of architects, who between them are responsible for planning and conducting the bulk of current building activity year by year, hardly consider steel at all for this purpose, or only very incidentally, are we to regard this as being in the nature of things, or as being due to a pretty well united prejudice against steel or on a poor opinion of the technical and economic potentialities of steel construction?

If it is really felt that the practical difficulties which up to now have prevented more extensive utilization by architects of steel for normal building jobs can gradually be overcome, until steel construction can compete on a fully equal footing with the other modes of building (this is the vital point!), then the great object must be to get more architects than before to some extent trained in dealing with steel.

With architects who are already working away in accordance with their own established usage (and consequently not too keen, out of caution if for no other reason, to experiment with unaccustomed methods), the possibilities in this respect are naturally limited; the more so as at present very little professional literature is available on the subject, and the individual architect usually only has a pretty vague memory of what he learnt in his student days about building with steel. Many architects however would probably be more receptive to the idea of using steel if something were done to bring home to them its present-day potentialities (particularly in connection with modern connecting methods, such as arc welding), in a way that fits in with their own special knowledge.

Specific problems also arise in this connection as regards the training of architects. As long as only a few firms of architects concern themselves to any great extent with steel, it will continue to be treated in training rather as a subsidiary aspect or a specialized subject. In fact, it would be rather unreasonable, as things now stand, to devote large slices of the already crowded syllabus to the inevitably difficult and demanding subject of steel construction, as more than four-fifths of the profession can expect never to have anything to do with it again.

The steel industry's wish then to see more steel used in ordinary building is not such a simple matter as all that. Quite apart from the gradual elimination of existing practical obstacles, the main task will be to devise ways of getting the architectural profession properly informed and interested.

Georges MOISELET

Vocational Training of Architects and Engineers

(Original text: French)

I feel that, apart from the actual subject (improvement of conditions on site) several speakers have stressed a human problem with which the future of steel construction, and so of steel itself, is very much related. I refer to the problem of educating architects and engineers to deal with this method of construction. In my view we should hope and pray that before long there will develop a species that is a cross between an architect and an engineer.

But before that happens, if it ever does, could we not try to promote the training of selected architectural and engineering students who would spend a period at a college being instructed in the theory of steel construction?

This is not a new idea, but I do feel there is something in it.

As regards my own country, France, I presume it would be possible, given the willingness to incur a certain amount of expense, to take a number of young men each year from the technical training colleges (Ecole des Arts et Métiers, Ecole Centrale, Polytechnique and so on), plus the same number of architectural students and have them work together for a time to turn them into real teams.

It would be a question of slow nurturing, but, one has to accept that in dealing with people, things do take quite a time, whilst time passes quickly.

I feel certain that at the end of some years we would thus have produced teams who could set the general pattern, and above all could work together.

Ernesto RULFO

Economic Aspects Involved in the Planning and Execution of a Project

(Original text: Italian)

A great deal of attention is being devoted at this Congress to the organizational problems of the customers, that is, the steel consumers; special emphasis is being laid on the dualism between building in concrete and building in steel, as seen by the architect, the designer, the engineer and the contractor.

May I point out, however, that nothing has been said as to the economic side of planning and carrying out a project.

The difficulty of assessing the cost of steel, and obtaining information from our suppliers' price schedules, is so great that it is very often a matter of guesswork to establish the relative merits of two or more alternatives.

The proportion of steel used in a reinforced concrete structure will of course be different from that used in an all steel structure.

Therefore to my mind it would be desirable for a scheme to be organized, under E.C.S.C. auspices, whereby a modernized form of training would be provided for senior sales personnel throughout the iron and steel industry of the six countries. The courses would be of approximately ten days' duration, to be held two or three times a year. They would be addressed by experts on business organization and sales promotion, from universities, research centres, seminars and so on. Direct co-operation between these lecturers and those attending the courses would make for a progressive homogenization of steel marketing and handling methods.

Special attention would be usefully given to the following subjects :

(1) market studies;
(2) price policy;
(3) sales organization;
(4) advertising;
(5) after-sales service to consumer;
(6) training of sales personnel.

It is often difficult to carry out the E.C.S.C. Treaty owing to the differences in sales systems among the six countries. The marketing patterns in the steel industry vary so widely that it is high time we had some uniformity, some homogenization, and harmonization in the conditions of sale, so that the cost of using steel will be comparable as among the member countries.

As a result of the lifting of tariff barriers, steel consumption is able to operate on the "communicating vessel" principle. Therefore it is essential that, the designers and the purchasers know the price of the steel delivered to the site (which varies according to availability and origin). The designer has also to know what difficulties he is liable to encounter in selecting one type of steel as against another with regard to prices, availabilities, supply problems and delivery dates.

Therefore it is essential to study all practical data available, which can be easily referred to. In other words, what is needed is to line up conditions of sale and ensure market transparency, to enable the buying and utilization of steel to be properly regulated.

The point is of course to avoid untoward and unexpected developments which could interfere with our aim, namely to increase the use of steel.

Rosario MASSIMINO

Prospects for the Popularization of Steel-Framed Structures in the World Market

(Original text: Italian)

We in this field are still in our nonage, not to say in the nursery. We certainly cannot compete with traditional reinforced concrete construction, which has developed through a whole series of stages, experiments, changes in building methods and building systems, and more particularly in systems of assembly and erection.

Today we are having to compete, at a considerable disadvantage, with this leviathan. Nevertheless, we did yesterday receive a message of encouragement from the President of the High Authority. He told us, encouragingly, that it was our clear and bounden duty to ensure the use of steel on a much wider scale. Now however steel is not in short supply within the Community, but quite the reverse : our production exceeds the consumption potential of the Common Market.

The third countries are emulating us with the greatest determination. We need to plan, to review the position and to see how and when this increased and increasing output of steel from all the E.C.S.C. countries can be employed : we need to find other fields of utilization for our basic product ; further outlets and uses for the steel we produce.

Today, the prerequisites are present for the more extensive use of steel in building. The labour market is affected by the social conditions now prevailing ; we are faced with rising labour costs, and at the same time with an increasing shortage of workmen, since building is not the most attractive of jobs from their point of view. In other sectors — such as in mechanical engineering, the chemical industry, the oil industry,—even the mechanic gets higher pay and has easier working conditions. Therefore it is necessary to try to use as little human labour as possible in building, and consequently prefabrication with steel structures is an absolute requirement which we must meet, this is in our own interest as steel producers. We cannot at present say to the contractors, or, more important, to the clients, "Use our steel structures for your buildings," when these steel structures cost more than the traditional ones tried and tested over the past two hundred years. We must compete in a field where economic savings can be made (a)—on the overall estimate for the job, and (b) in building time. All this presupposes thorough co-ordination between the items going into the constitution of the steel structure itself and those connected with its assembly and erection.

The prefabrication operations are closely interrelated, and it is impossible without prefabrication planning to turn out really accurate framed structures. The latter, as has been repeatedly pointed out, account for 15-20% of the total cost of the job. If we treat the matter under two separate headings, first on the load-bearing structure, we shall not get a true economic picture. What we have to do is to view the building process as a whole, from the economic angle, the angle of the total and final cost of its execution. Unless we do this, we shall achieve nothing as regards exploiting the potentialities of steel frame construction, because nobody will consider buying buildings of this kind if they are more expensive than buildings with a load-bearing framed structure in reinforced concrete.

I feel, therefore, that the matter is a complex one, and that careful consideration should be given to this morning's proposal for the establishment of an E.C.S.C. committee to go thoroughly into this intricate problem of harmonizing all the elements involved in the process of steel frame construction. One reason why we should devote detailed study to the question is that the building regulations of the individual E.C.S.C. countries differ considerably, and each country has its own established customs, and indeed its own requirements.

The problem of harmonization as regards assembly and erection methods is exceedingly important, and to my mind fundamental. In Italy we have had a real opportunity to act in this connection, as we all belong to the same group of firms, some of which concern themselves purely with assembly operations, while others are engaged in prefabrication of structures, and are producers only, leaving the former to do the assembly and erection. But to me it is vital that one firm should be responsible exclusively for the planning and fabrication of the steel structures, and another for the work of assembly and erection, preferably not only of the load-bearing structures themselves, but also of the finishing work. In any case there should be a single body dealing solely with this part of the operation.

Jacques BENDER

Industrialized On-Site Assembly of All-Steel Buildings

(Original text: French)

Following the adoption of steel building techniques, we have proceeded with industrialization both :

(a) at the works, where the components for the entire structure are mass produced, and components, no matter the type of building ;

(b) on the site, where all structures, of whatever nature, are assembled with screwdrivers and box spanners. (1)

I felt, therefore, that it would be worth outlining to the Working Party what we have achieved in this connection.

At the time a project is drawn up, a list of items required is compiled from special coded plans, which are virtually photographs of each projected storey. The timetable for stocking, for delivery to site and for assembly is laid down in detail (2). Assembly times are fixed with precision and checked daily. The allocation of special teams for each particular job involved, with the appropriate light equipment, helps to ensure that operations proceed smoothly, quietly and tidily.

Here are the first results of these arrangements :

(1) a five-storey building 80 metres by 17 1/2 metres (approx. 260 x 58 ft.) was erected in one month by very young personnel (students from a technical college), who kept perfectly to the timetable; (3)

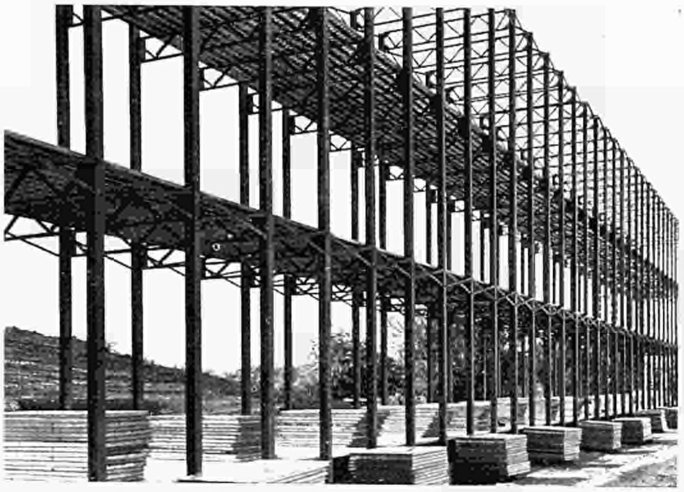
(2) a two-storey building of 3,000 square metres (approx. 10,750 sq.ft.) was erected in a fortnight and finished (all trades included) in a month.

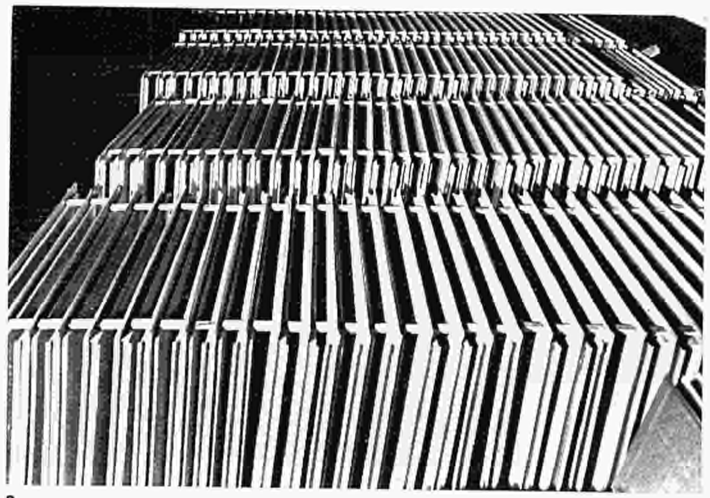
The assembly average is fixed at 15 spans per day for the bolted framework and at 5-10 per day for the wall panelling. The span referred to is the full span from ground to roof on both sides. The individual components never exceed 60 kg. (approx. 130lb.) in weight.

Rational site organization is absolutely essential to successful industrialization. In addition, it soon produces a considerable social improvement since it becomes possible quickly to secure a corps of specialized workmen who can readily be induced to try to outdo one another in performance.

The use of steel will thus enable large numbers of buildings to be constructed without the need to draw on traditional skilled labour, which is steadily falling off both in numbers and in quality.

On a site so organized, a team of 20 can build a technical college of 7,000 square metres (approx. 25,000 sq.ft.) in 6-7 weeks.

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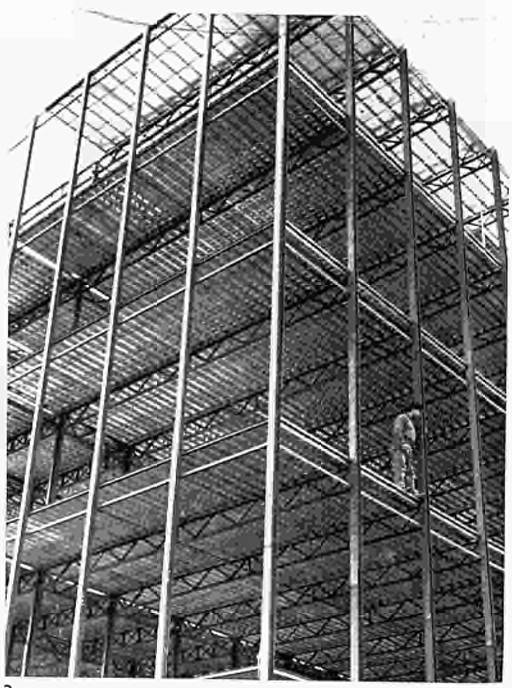
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Willy JURISCH

On-Site Rationalization

(Original text: German)

I would like to say a few words on the subject from the point of view of the Erecting Engineer. We on the assembly side too have to concentrate all the time on keeping our operating costs low, and we very much appreciate the start that has been made here in comparing notes on the matter.

On-site rationalization of steel construction during the last twenty years has unfortunately not shown the same progress as rationalization at the workshop end. On-site costs are a substantial factor, amounting to anything from 10% to 100% of the price ex works. In the case of the construction of long span bridges, hourly wage costs on the site are often as high as those in the workshops.

How can we manage to lower our on-site costs ?

(1) By differentiated, flexible planning of the assembly operations, as a preparation. In this I include selection of the most suitable type of assembly, planning of the course of the operation, with all the equipment and auxiliary installations, delivery schedule, dispatching schedule, detailed assembly and erection timetable, labour deployment plan, and so on.

(2) By using modern, wage-saving appliances and machinery. This involves quite considerable capital expenditure. An equipment pool for assembly firms in a given area might make for more economic utilization and less depreciation of equipment.

(3) By efficient site organization :

(a) good overall co-ordination among all the firms associated with a building project;

(b) good site organization, together with constant checking of costs and times, to enable any stoppages or mistakes to be effectively dealt with.

(4) By designing with convenience and ease of erection in mind.

(5) By improved site training for executive and specialist personnel.

Here are one or two comments on each of these points.

(1) Assembly preparation requires specialized assembly engineers with comprehensive training and wide experience, accustomed to thinking in terms of cost. The work done in this regard must accordingly be in line with the scale of the project and with size of the steel construction firm. Moreover, not all small and medium-sized assembly firms have enough specialists of this kind available.

In the United States, the Soviet Union and certain other highly-industrialized countries, large area-wide assembly concerns which operate for quite a number of supplier firms have been set up.

(2) A good many steel construction firms are not able to make full economic use of a full-scale modern equipment park. In Germany this situation has recently been turned to account by firms which have been set up to hire out power cranes. In my view, it would be possible to reduce costs by instituting area equipment pools for steel construction firms, or big assembly concerns of the type I have mentioned,

(3) Both the clients and the firms carrying out the project would benefit greatly by intelligent co-operation.Every effort must be made to eliminate auxiliary process time, and waiting time on sites.

4) Weight is not the only consideration in connection with lowering of costs. With wages rising on every hand, we should concentrate on simplicity of design involving as little expenditure in wage costs as possible. The trouble here is the very widely varying, and in some cases decidedly excessive, requirements of clients as to tolerances. Further and considerable savings could be achieved in this respect by standardization and appropriate design.

(5) The Assembly Problems Committee of the German Steel Construction Employers' Federation made a start some three years ago on providing practical training for assembly foremen and junior assembly engineers : weekend seminars are now successfully organized once a year in a number of major cities, and at these careful attention is devoted to safety on assembly sites.

To sum up, there is a great deal that can and, in the interests of increased steel utilization, must be done to improve productivity on building sites. There must be less improvisation : we must plan more, reduce auxiliary process and idle times, improve site organization, design with a view to ease of assembly, use modern, labour-saving equipment, and train better executive personnel and specialized tradesmen.

Enrico DE SMAELE

(Original text: French)

One way of overcoming the problem of harmonizing the different elements involved in building a steel framework structure might be to have temporary partnerships between framework fabricators and general building contractors.

I mean temporary partnerships, and not just collaboration, for the economic success of the project depends very largely on the organization, both of the fabrication, assembly and erection of the framework, and of the contractor's methods with regard to the installation of floors partitions and walls, service-core installations and so on. Each operation is highly dependent on the other.

Obviously, the introduction of metal framework in building requires a change in the whole mental approach of the contractor. Prefabrication demands a wholly different psychological and practical preparation both of the men taking part and of the techniques they employ. Such a change would be advanced by the regular co-operation of the trades mentioned.

Hendrik BLANKENSTIJN

(Original text: Dutch)

I belong to quite a different side among those represented here—to what we call in Holland the commissioning builders. In Holland the local authorities are among the biggest clients of the building trade. They have had first to be convinced that traditional building methods could be largely replaced by industrial methods using mainly concrete components or pre-cast concrete.

In Holland, and in other parts of the Community also, the great question is how to build more, and how to utilize idle capacity. At the present Congress I have gathered that the steel industry is in a position to help us with our housing problem. Well, then, you will have to put your case convincingly to the client (usually represented by his architect). That will definitely mean making a push, just as the concrete industry is doing. You will have to offer fully worked out plans, with firm and final figures, showing that it is possible to build more by expanding traditional capacity with the aid of the steel industry. You need to prove that you can build faster, that your results will certainly not be poorer in quality but rather better, and—the difficulty we are constantly encountering—that this will cost not more, but possibly less.

Actually this is not necessary. I know that in Holland when there are projects that will initially work out rather more expensive but will give us more houses, there will not be any haggling over minor cost differences.

I feel you should contact not only the architect but also the local authorities, and should help the central authorities by furnishing complete plans on not too large a scale. Look at the E.C.S.C. experimental schemes : they ought not to be confined to housing for workers in your own basic industries, they ought to be put up to the Community countries for everyone. Put forward your projects first and foremost in your capacity as main contractor. Show what you can do in the building line !

Findings

The first speakers highlighted the need for undertaking the study of productivity on steelwork erection sites within more general terms of reference. In actual fact, this stage of operations is an integral element of an overall process involving a number of decisions depending successively upon the investor, the building owner or sponsor, the architect, the consulting engineer, the main contractor and the steelwork contractor. It thus appears essential to examine the effect of these various aspects upon the erection site problem properly so called, inasmuch as those aspects constitute so many determining factors in arriving at an optimum solution, *i.e.*, a solution that is economical to the purchaser. It is within the scheme defined by these determining factors that the steelwork contractor should find the means of increasing the productivity of his site and thus turn the possibilities of steel to good account.

1. Starting from the foregoing considerations, the Working Party came to the conclusion that steelwork contractors should be encouraged to give fresh thought to the problem of the structure of their branch of industry. Some of these contractors will find it advantageous to specialize in the prefabrication of components or sub-components; others will have to consider extending their activitites so far as to include the whole of the structural engineering work, so as to bring certain factors under their control which ordinarily elude them.

2. It appears essential to intensify activities aimed at developing contacts with initiators of projects, such as architects, consulting engineers, and public authorities.

3. Once he has been included as an integral part of the overall process, the steelwork contractor must strive to find the most economical project, *i.e.*, taking account of the expenditure in respect of fabrication in the works, transport and erection. It too often occurs that only one of these aspects is considered, whereas actually it takes a harmonious combination of these three aspects to produce the optimum solution.

4. The statements by the speakers showed the considerable importance that must be attached to giving wider currency to planning methods. These are the essential guide to enable a job to be carried out within the time allowed and within the limits of a predetermined budget.

5. Such planning should more and more be based on precise data concerning the time devoted to various site operations. In this respect it appears necessary, in order to collect such experimental data within not too long a time, to promote collaboration at international level. It would therefore be desirable for the E.C.S.C. to encourage the setting-up of a Working Party which could reach agreement as to the methods of observation, so as to facilitate the application to a great many construction sites throughout various countries, and which could, on the basis of those observations, establish standards related to a common yardstick of evaluation, *e.g.*, the working hour.

6. The Working Party calls attention to the fundamental problem of the professional training of engineers who, at present, do not appear to be able to apply these new methods. This is more particularly true of staff engaged in job planning, having regard to all the implications mentioned above, contractors' agents who often fail to perform their task as site manager, and the engineers who have to maintain liaison between the works and the construction site. Here, too, the Working Party considers that the E.C.S.C. could very usefully promote the training of such staff.

7. The Working Party's attention was also called to the problem of the safety of erection personnel and, consequently, of the importance of active medical supervision of the men on the site. Suitable publicity might have the desired result.

8. With a view to bringing the new site organization methods into general use, it appears desirable to encourage steelwork contractors to make erection work an integral part of their activities. Apart from the large contracting firms—whether they be fabrication contractors or erection contractors— that can develop this new side to their activities as a paying proposition, however, it appears desirable to encourage the setting-up of pools of erection firms, which could thus operate on a worth-wile scale and could serve the needs of small and medium-sized steelwork fabrication firms.

9. In order to make available to steelwork erection contractors full and precise information on the technical features and the cost of utilization, it would be desirable to encourage the complication of central card-indexes.

The Working Party found that, in order to promote the increase of productivity on construction sites and thereby to widen the scope for the use of steel, it was necessary to take all the above-mentioned facts into consideration. Since a great many problems could not be given due attention in the Working Party's proceedings, it appears desirable that these problems be further dealt with by appropriate groups, *e.g.*, the national professional organizations and the Convention Européenne des Associations de la Construction Métallique (European Convention of Structural Steelwork Associations), which is, indeed, already engaged in looking into some of them. Obviously, the patronage of the E.C.S.C. would give those groups some very helpful encouragement and a measure of authority that would be justified by the economic importance of the matter being dealt with.

4 *Summaries of the Congress Proceedings*

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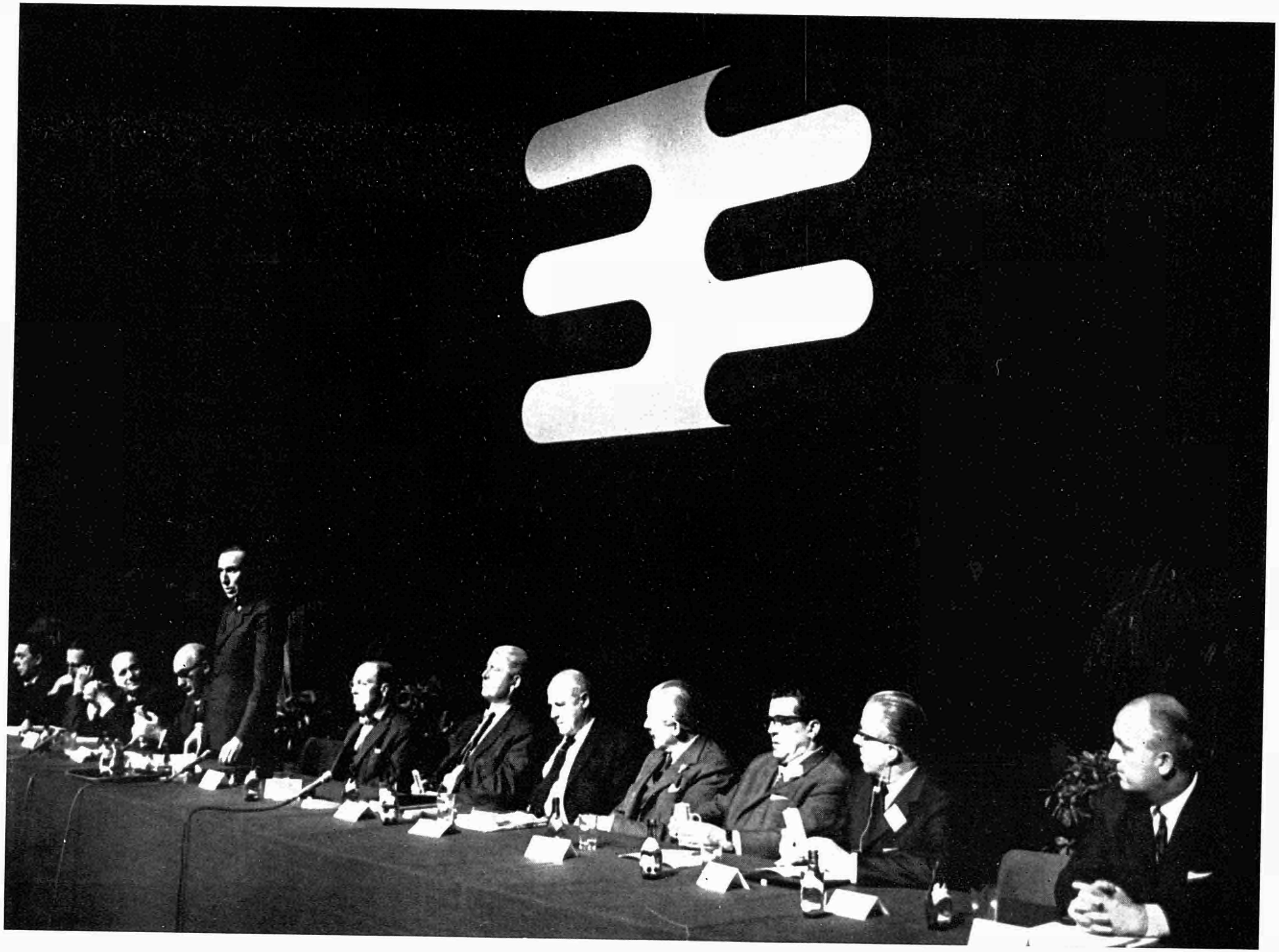
Closing speech by the President of the High Authority



Egon EIERMANN

Vice-Chairman

I have been asked to address you on the findings of the Congress. Well, it is no use expecting me to do that, because I obviously could not have been at all the meetings of all the working parties. And I particularly don't want to have to comment on the scientific contributions, because formulas and tables always give me the creeps. I am speaking to you as an architect, giving you my personal reactions on what I have seen and heard.

Steel is to architects the object of their often unrequited devotion. Steel constructions demand the highest professional attainments; they demand logical clarity down to the smallest detail, and classical sobriety manifesting itself not least in the employment of the right-angle as the *optima ratio*.

Well, today architecture has fallen for concrete in a big way, producing a positively baroque exuberance of form more like statuary than building. Architects will know what I mean.

But one striking thing at this Congress has been that the architects congregated chiefly where the subject was the opposite, namely rationalization and standardization, where among other things we got that extremely sound paper on prefabricated steel structures by M. Jacques Bender, of Paris.

As a lover of steel myself, I may tell you that to me steel construction represents the aristocratic principle in building. None of your mushy stuff squeezed out in strips, pushed and pulled this way and that, gradually allowed to harden, and only then given its stiffening of steel.

But what I find most absorbing, what really attracts me wholeheartedly to steel is this: steel is removable.

It appals me to look at all these concrete bunkers and blocks and know they are there to stay. Steel does not go brazenly claiming permanence for things that have no business to be permanent, and for that I love it. It satisfies a high ethical and aesthetic sense in me which as an architect I extend to the material I am working with.

That is a preliminary confession I feel I have to make to you.

For these reasons, I want to show you steel in its utter purity. From that utter purity the Eiffel Tower and the great bridges we know derive their beauties. It underlies the design of the French and German pavilions at the Brussels World Fair; the design, too, of John Deere's office block which Mr. Danforth has just shown us; and the design of the new premises for the German Embassy in Washington.

In these buildings, the structures appear in their natural shape. The supports stand free and visible in front of the wall. Scientists and engineers ought to look at things like these, to sense the progressive spirit in which architects are working for steel.

But I will stop philosophizing and get down to realities, to the two problems even this Congress cannot solve for us architects—coating, that is to say material maintenance, and fire prevention.

Dr. Kollbrunner, with a definiteness that leaves no room for doubt, has given us the formula for fire risk exposure. Unfortunately, the position in practice does not correspond at all. With us, any fireman can fix whatever requirements he sees fit. And he does.

When one of our speakers shows us a multi-storey building and says the structures do not need to be covered because it is for an exhibition, well really, that is giving the authorities a pretty free rein for interpreting and construing regulations as they please. As if the building could not have gone on fire during the exhibition!

But nothing is more prejudicial to steel superstructures than regulations which cease to be regarded as binding because they are obsolete and wrong.

If when Committee III of the European Steel Convention completes its work the High Authority were to get these provisions really made law, it would be an enormous help. I sometimes wonder what there is left to burn in these days of fireproof flooring, and steel door-frames, and buildings with no timber in them. We are compelled, and in duty bound, to demand the revision of all these out-of-date regulations from the last century if we want to build in steel today.

You do not get actual flame, you get smoke, and as we all know the majority of people in a fire are not burnt but asphyxiated. Right: may we please know soon when the new fireproof coatings are going to be officially approved? And whether they do their job? We are just burning to find out. That is the kind of thing architects have to know for building in steel.

And the second problem, coating.

When Mies van der Rohe, whose buildings the whole Faculty of Steel Construction ought to study as a cultural milestone for the world, was asked by my students — funnily enough it was the first thing they did ask — what he would do to protect steel, he replied in his pithy way, "Paint it". I don't think that is enough. All right, we sand-blast it and we cold-galvanize it. Good thing for us that can be done, even if it is not such a good thing for the men who have to do the sand-blasting.

Now we hear from America of a new process, used incidentally by Saarinen on the Deere building I referred to just now: it seems decomposition completely stops after two years' rusting. If this is really so, it is more important than any new development on the producer side, unless we were to get weatherproof steels at affordable prices. Production of Corten steel over here, for example, would help tremendously.

If the High Authority turned its attention to these matters, and provided substantial funds for research — I mean on steel utilization — we would not have to worry.

When I hear the new series of wide-flanged beams offers increased flange thickness with no shortening of the web, I can tell you right away that used "straight" they may be a help to the engineer with his pure calculations, but they will not help the architect in the slightest. But nobody asks our opinion.

Since steel started to lag behind concrete, almost the same has been happening as with coal and oil. Now that prefabricated concrete components have been developed, together with a very simple method for connecting them, I cannot see why steel does not also experiment, this time following concrete, in order to evolve fixed connections not based on screws and rivets and welding. I would like to remind you of the, at first glance, highly theoretical work of Wachsmann, which, significantly, is known to every architect, but hardly to a single engineer.

I do not know whether it would work, but why is not anybody trying to make it work?

For as long as the steel federations leave the architect to dream up ideas on his own, I do not think much of the prospects for increased steel utilization. Take the various efforts that have been made with prestressing, a process for which steel is eminently fitted, to arrive at new, vital forms and an unexampled economy, including economy of style and line. We have to admit that concrete has shown to better effect.

Now a personal comment on composite building, *Mischbauweise*. The German *mischen*, "to 'mix," "to cross," suggests a mix-*up*: you "mix" (shuffle) a pack of cards. To *mischen* or mix, to obtain a composite, is to combine two unconnected things. When I look at the thousands of welded studs transmitting the stress from steel to concrete, I cannot think the method is ideal. Economically it may be all right. But — how shall I put it? — it does not *fit*. A steel roof on the steel structure would fit. But what steel roofs are there? Only those extremely costly American roofs? And on that point I would ask: Are there any exploratory studies on the steel concrete price relation that do not give the preterence to steel, having regard to its lighter weight, and the importance of the time factor in assembly, and the fact that the rest ot the operations go faster? Or is that, too, something architects have got to do for themselves?

Anybody who has ever built a multi-storey block with a concrete core knows the maddening refusal of the concrete to connect precisely with the steel. If you must use concrete, do at least have the prefabricated concrete component fitting the prefabricated steel component. Consequently, of the composite structures we have been shown the one that most impressed me was the hall with the north-light roofs consisting of big prefabricated concrete units mounted on the load-bearing steel. That was homogeneous, because the two materials were used in accordance with one law. And these considerations are going to become still more pressing when in course ot time we find ourselves having to deal with steel-plus-plastics.

So I am constantly being confirmed in my opinion that building is an aesthetic and intellectual business, in which the sciences, as Prof. Stüssi has said, only lend a helping hand.

To sum up: so far as I am concerned the Congress has posed questions rather than solved them. It has also produced this truth for the architect. To have steel utilization, you have to have steel production, and you have to have possible applications for steel. And if its applicability is impaired or destroyed, or even just obscured, either by insufficient attention to essential details — weather-proofing, for instance — or by out-of-date regulations, then there is no point in trying to boost increased steel utilization.

These few days in Luxembourg have been most stimulating and worth while. We have all met friends of ours and got to know interesting people. We all know how valuable human intercourse is, both for now and for the future, in the matter we have been dealing with. For this aspect too, for the conversations and exchanges of ideas we have enjoyed, I and all of us owe the High Authority a real debt of gratitude.

Max BAESCHLIN

Vice-Chairman

In the course of this first European Congress on Steel Utilization, I have been much struck, not only by the extraordinarily objective accounts given of the many and varied potentialities, and occasional deficiencies, of steel and steel construction, but more especially by the radical trend in favour of the industrialization of building that has emerged from all the papers and motions. This seems to me of altogether fundamental importance. It is my firm conviction that this revolution will be one of the most momentous in the history of building.

With the population increasing by leaps and bounds, and with the rising standard of living and the great strides being made in technology, brisk activity is in progress in all parts of the steel construction sector. The aim may be defined very simply as being to build more quickly and more cheaply, but the means are new — industrialized building, based on the utilization of all the facilities afforded by modern technology. I have been asked briefly to recapitulate and sum up for you the ways and means, and also the difficulties, involved by this new trend. However, it would be absurd to suppose that I could possibly touch, even by implication, on all the tremendous number of excellent and valuable points made by scientists, architects, engineers, and economists too, in the past two and a half days. To sift this fund of thought would require more leisure for preparation, and moreover I should considerably exceed my allotted time for addressing you, which I am anxious not to do.

I propose simply to select one or two problems and ideas which I feel to be significant.

Thanks to the progress in steel production in the last twenty years, we have available for constructional purposes a large variety of steels, which intelligently used can ensure substantial savings in steels consumption. The new series which are being turned out, offering a wide range of cold-rolled sections and welded hollow sections, are enabling those building in steel to develop all kinds of different types of structure.

It is up to the architect and engineer to use these opportunities in order to develop new architectural and statical forms, which will in turn be an incentive to the steelmaker to produce more new steels and more new sections.

In theory all this sounds very simple and very nice. But as the men on the job know, there are still certain limits to such collaboration, especially between architect and engineer.

To us steel construction men the architect is a very important figure. He is not only the spiritual father of the structure, but also the client's representative, and so able to guide the choice for or against the use of steel.

Many, indeed I think may say most architects fight rather shy of steel construction, because they do not know enough about it. The fault lies partly with the still somewhat imperfect training arrangements for architects: the technical colleges teach them too little about the possibilities for employing steel in building. In addition the steel construction specialists themselves are to some extent to blame for architects' lack of enthusiasm: they have confined themselves for too long to problems of statics and design, and dismissed as a mere detail the whole subject, so important to the architect, of the combination of other materials with steel.

To make the most efficient use of the steels available, it will be necessary to secure the revision of obsolete regulations, and also to work for the international unification of standards. It is urgently necessary for European steel construction that there should be uniformity on methods of calculation, especially in respect of thin-walled sections. Some preparatory work has already been done in this connection by the European Convention of Steel Construction Federations, and it is an open secret that the way is hard and stony and sown with prejudice.

A good deal of crass ignorance still exists with regard to the assessment of the fire hazard in steel buildings. So I consider it to be one of the most important tasks on the steel construction side to make known the latest research results on the fire load in modern structures and the extent to which it governs the duration of combustion. It is encouraging to note that efforts are being made in this direction in many countries, with promising results. It seems to me obvious that the influential support of the High Authority would carry them speedily to complete success.

Another problem in steel construction, corrosion-proofing, is today well on the way to solution: we are able to provide suitable protection against corrosion by means of various kinds of surface treatment and preservatives, according to the purpose of the structure concerned. But for so long as we are compelled to give special additional protection to steel the in many cases solid wall of prejudice against steel and steel construction will persist. It is therefore absolutely essential that research in this connection should be pursued and extended, and that endeavours should be made to reduce the cost of the existing corrosion-proof steels — more especially since no economic method of corrosion-proofing thin-walled sections has yet been devised.

In discussing the material for steel structures today, it is no longer possible to think purely in terms of steel itself. The standardization drive covers everything that goes into the making of the completed structure, and for economic reasons it will probably not be possible to dispence with other materials for quite some time.

One point I may mention here is the question of flooring. We must particularly welcome the efforts in progress to develop a combination of shaped steel sections with a concrete slab, furnishing a cheap and easily removable floor.

Similar efforts at cost reduction need also to be made for external and internal walls, account being taken in the design not only of soundproofing and temperature control requirements, but also of the piping and wiring to be installed.

Economic building does not depend simply on efficient utilization of the material available, nor even on steady progress in the development of really appropriate forms: the breakthrough comes only with the introduction of standardization and prefabrication. These are already recognized as basic to all progress in building. Thanks to the excellent work of a number of eminent architects, they are no longer synonymous with monotony: they have become a new constructional mode of expression. The reasons which impel us to use the fixed or the adjustable type—French has the neater expressions "*préfabrication ouverte ou fermée*" — depend simply on the functional demands of the structures.

One essential requirement for the success of the great prefabrication drive is to have uniform bases. If prefabrication is to be the fundamental characteristic of the new style of building, it must be applicable throughout the construction right down to the smallest detail, and be based on a given system of moduli. I consider it vital to put a stop to the eternal battles in this regard, and introduce a uniform system of moduli for building everywhere.

The builder in steel today is more and more having to accept that he must revolutionize his previous fabrication methods. Even efficient standardization and consequent production in large series will not have the desired effect unless workshop fabrication is modernized, mechanized and if necessary automated.

It will cost a great deal of money to effect the necessary conversion of the steel construction firms from artisan to industrial methods. Yet even an automated installation cannot meet every demand that might be made on it, and hence has to be adapted to particular products. Accordingly, the builder in steel must scrap his old and perhaps cherished idea that he can and will do everything: he must concentrate on specific fields, restrict his range of fabrication, and fully accept the modern industrial approach. This is no easy matter, but it has got to be done, particularly as he will ultimately, both in the narrower and in the broader building field, come to perform a different function than he used to. He must evolve into the general co-ordinator who provides completed constructions ready for use.

The conversion of the steel construction firms to industrial fabrication will also be of direct assistance in reducing the cost and time of assembly and erection. Only standardization will enable the problems to be properly tackled and the existing assembly and erection methods improved and simplified.

It is already clear from the Congress reports that in the steel construction sector all the mental and material prerequisites are pretty well on the way for the coming plunge from operation as a trade to operation as an industry.

The Congress now closing has aroused very wide and sympathetic interest indeed in specialist circles. To my mind, only such a gathering as this that the High Authority has mounted and organized is an adequate occasion for conveying a full picture of the latest technical advances, and at the same time comparing consumers' needs and producers' potentialities. We owe the High Authority a debt of gratitude and appreciation for its action; to keep up the contact here established, it would be well for other such congresses to be held at regular intervals.

Jean-Marcel JEANNENEY

Chairman of the Congress

The criterion of a congress's importance is not merely the number attending it—in this case, the number exceeded all expectations,—but also, and more notably, the newness and pertinence of the papers presented, the liveliness of the debates in committee, and also the give-and-take of opinion in individual encounters outside the actual proceedings, sometimes the most stimulating part of the whole occasion.

I can speak as a neutral, since I was not in fact personally involved in these intellectual exercices. So it is in order for me to say that the Congress rates very high in all these respects, and I would thank the appointed speakers, the chairmen of the working parties, all those who contributed to the discussions, and most especially the two Vice-Chairmen who have just, as it were, distilled for us the philosophical content of the proceedings generally, for all their good work in this most necessary cause. But you will agree that I must also express thanks and congratulations, this time on behalf of you all, to the High Authority — first of all to President Del Bo, whose original idea it was to call this great world Congress, to Dr. Fritz Hellwig, the organizer and moving spirit at High Authority level, and, if I may, Mr. President, to your Director-General Signor Peco, the pivot on whom the practical arrangements revolved.

The avowed aim of this Congress on Steel Utilization was to open up new outlets for the steel industry, whose position Dr. Hellwig so ably outlined for us in his opening address, stating that some concern might be felt as to its future markets and hence its equilibrium.

Hearing and reading the various points that have been made, I have recognized, as you have, that the effect of some of the advances described or demanded will be, or would be, to reduce steel consumption. When our engineers labour to calculate strengths more accurately, when we hear that this or that customary safety measure is unduly strict, when we are shown how new welding processes will make it possible to employ much thinner plate or sheet, it is obvious that technological progress such as this means that less steel will be consumed.

Now, does this imply, Mr. President, that the people who devised these processes, or came here to tell us about them, are letting down the side? The answer is, certainly not: it is by enabling the same result to be achieved with less steel that we can make the use of steel a more paying proposition. In this age of competition, when all materials vie with one another to perform a given service, what will bring steel out on top is not only its inherent, its technical virtues, but also its economic virtues — that is to say, its cheapness to use. It is no paradox to say that the less steel needed to achieve a particular object, the more steel will be bought.

But the great point of this Congress was to survey the fields in which more steel could be used, and to discover new ones. In this regard the various uses here reviewed are not of comparable importance economically.

To the engineer, to the technician, and also to the layman who likes to see what the human mind can devise, improvements in the building of temporary roads, or in bridge-building, or in factory building are of the highest interest; they are, too, certainly of some value from the point of view of steel sales, but they do not compare, to my mind, with the potentialities in the field of housing. Now there the openings really are tremendous — practically unlimited, and growing all the time. We have all admired the slides which were shown the day before yesterday and this morning of the marble halls of a few great companies. Bravura pieces without a doubt, but myself I would class them with Italian *palazzi* of the sixteenth, seventeenth and eighteenth centuries, or the palaces of European kings. They are *sui generis*, expressions of human genius which are of necessity for ever unrepeated and alone, serving perhaps as pilot projects for new techniques, but always, inevitably, in numbers the merest handful.

But consider the hundreds of millions of human beings all over the world, even in our own affluent countries, who are housed poorly or long to be housed better. We of the developed world, after a period of Malthusianism, have reverted to multiplication. Our populations are growing apace: yet it is not only that which is pushing up housing demand and will send it soaring, it is also the rising standard of living. That rise is reflected first in better feeding; then in the acquisition of motor cars and refrigerators and television sets; but ultimately, when all these needs, for essentials and for luxuries, are more or less met, its main effect is to make people concentrate on better living conditions. Better in that they are more spacious, more comfortable, less exposed to heat and cold, less exposed too, I would add for the attention of the architects present, to noise from outside. The demand is pretty well unlimited. Yet, whereas the last hundred years have witnessed astounding increases in productivity in industry and in agriculture, when we think of the process of building a house we still visualize a barrow, a crane, a concrete-mixer, and usually rather shabby workmen toiling away as best they can with raw materials in the wind and the rain. I am a layman, I know, but I definitely think housing is a field in which we might see tremendous strides in the next ten or twenty years. Let us face it: there is a very serious lag between the technology by which we build supersonic aircraft and send rockets to the moon, and the technology by which, generally speaking, in most countries we build the majority of our houses.

I firmly believe a congress such as this can do a great deal in this connection. I do not love steel, as our Vice-Chairman has told us he does, but I do think — and still more after hearing the speakers at this Congress — I do think that steel more than any other building material possesses the qualities needed to bring about a breakthrough in our building methods, in on-site organization, and indeed in the design of our dwellings, a breakthrough that will combine the saving of manual exertion with the meeting of the basic needs of the human individual, who is not and should not be satisfied to occupy one cell in a vast featureless block, who is not prepared to live like a bee in a hive. Our architects, our engineers, our designers must contrive to reconcile the demands of productivity with the undeniable need we all have to follow our personal fancy now and then, to give our homes that individual touch that makes them different from other people's. I do not think that that is at all incompatible with the industrialization of the building trade. It may have been with an earlier and faulty process of industrialization, but by fuller study of the materials available, it should be possible — in fact architects tell me it is possible now — to work out all manner of permutations and combinations taking account of locality and climate and of what, rightly or wrongly, the people who are to live in these cells, themselves prefer and desire.

A civilization is essentially a spiritual thing. Yet no spiritual civilization, at any rate in our time, can last and evolve without a proper material foundation. And civilization is formed in the family. What conditions a family's life and development is its home, the framework afforded parents and children in which to live side by side and yet not on top of one another, to develop their respective personalities, and benefit by the best in the personalities of others, without being bulldozed by those they live with. Of course this does depend on each individual person's idea of what family life and relations with other families mean. But it depends also, perhaps more than they themselves realize, on what our architects and engineers are doing and will be doing.

All this is perfectly clear, indeed self-evident, and I apologize for stressing it so insistently. But there are a mass of difficult problems to be tackled. Technical problems, of course, but administrative problems as well: I think the little band of civil servants here with us must have felt their ears burning on a good many occasions during the discussions, when architects and engineers united in damning Officialdom as to blame for their delays and failures. One has to make allowances. It is only human to try to shuffle off responsibility; no doubt if this were a congress of civil servants dealing with building we should have heard some unkind remarks about architects. But it is a fact that Officialdom is essentially conservative, essentially cautious; officials are not as a rule keen to assume overmuch responsibility, and any change in the rules creates responsibility. True, it sometimes takes more time and trouble to amend an obsolete regulation, universally agreed to be by now altogether beside the point, than to put up a twenty storey building. But that should be all the more an incentive to us to urge everyone having any degree of responsibility concerning these naturally most necessary regulations to engage in daily self-searching, and to remember that it is a civil service's proudest achievement to be able when the time comes to abolish a regulation it originally established. In administration as in technology, simplicity is the most graceful thing of all.

Technical problems, administrative problems, and economic problems too. In building as in any other sphere, to opt for a particular capital project is to abandon thought of another. It is the eternal question of choice, choice of priorities as among different desirable projects. That choice is essentially a matter of policy. Yet, policy or no, it is a choice that can be properly made only after comparing many judgments, many opinions, many views, at national and at international level.

By now, our technology and our relationships and our civilization are so intricate that nothing can be satisfactorily decided, or undertaken, or achieved without co-operation among men of widely varying background, training, mentality and sympathies. The need to bring skill and knowledge and enterprise into a common focus has never been so great as today. This Congress has been keenly conscious of the fact. Does this mean that nothing can be achieved any more except collectively, by carefully-contrived combinations of men and organizations? I do not think so. I think that the major advances are still, as they have always been, the work of individuals. In the final analysis, it is individuals who make hay with settled custom and change the course of the future. Indeed, I might add that the individuals in question are not always the experts on the subject: the mavericks have their uses. But even though the individual is indispensable, he can in practice do nothing without the machinery of organization. Several working parties, I know, have stated their conclusion that action should be taken to establish common foci, to give organized form to the focusing process, whether nationally, or internationally, or within a given profession or occupation, or among several. One of the things this Congress will perhaps have achieved will have been to bring home more clearly both the greatness of the aims and the fact that to attain them it will be necessary to form and canalize a tremendous fund of individual and corporate good will.

Such, Mr. President, seem to me to be the net conclusions emerging from these three days you have arranged for us. Who knows, this Congress may be the beginning of something. In fact, something has already begin to develop during our time here — something as full of promise yet as delicate to rear as a newborn infant. That infant, Mr. President, I now commit to your care.

Dino DEL BO

President of the High Authority

I have to confess that we Members of the High Authority were looking forward with a certain human trepidation to the opening day of this Congress. But we were more justifiably apprehensive about the closing session, today, since we knew that the Vice-Chairmen's reports and the Chairman's summing-up would contain an expression of opinion on the action we had taken in organizing it.

Well, now we know that that opinion is at any rate a kindly one, even though, obviously, it has immediate and pressing implications for the whole established structure of our duties and responsibilities.

Before proceeding further, may I first discharge the very welcome task of expressing our thanks. First of all to you, Mr. Chairman, who in the course of the Congress, and especially in your closing address, have shown yourself to be not only, as we knew you were, supremely qualified in economic matters, but also possessed of those particular humanistic endowments that have enabled you to state to us certain alternatives, to advance certain suggestions, and above all to furnish us with a dialectical interpretation of the Congress's work which will be a direct stimulus to our studies, our enthusiasm and our endeavours.

I have also to thank the Vice-Chairmen, who have steered the proceedings of the working parties, summarized their conclusions, and generally made a particularly outstanding contribution to the success of the occasion. In addition, as you yourself, Mr. Chairman, have just mentioned, our thanks must go to those who have borne the main burden of organizing the Congress.

Our thanks, then, to my fellow-Member Dr. Hellwig, to our Director-General for Steel, Dr. Peco, and his subordinates, to the Press Office, and I would take this opportunity, too, publicly to thank all those other members of our staff, of all grades, who to help make a good job of a Congress organized at such short notice were willing out of sheer attachment to the European ideal to give of their free time, and even, on occasion, to go short of their sleep, in order to enable the schedule we had fixed to be strictly adhered to.

With regard to the opening ceremony, I must first express our humble thanks to their Royal Highnesses the Crown Prince and Princess for graciously consenting to honour the occasion with their presence.

Our gratitude is also due to the Goverment of the Grand Duchy and to the Corporation of the city of Luxembourg, who made available every facility and did everything in their power to help overcome the various unavoidable difficulties and ensure the participants of an enjoyable stay.

A special word of thanks, in addition, to the representatives of the Press, who have been able to record the progress of our work for the public in the six countries and beyond. Now inasmuch as it falls to me to draw certain specific conclusions from the Congress, I feel that the focus must rest more especially on one particular set of findings. Of course, the findings I refer to are not as yet fully developed, indeed in some respects not even definitively viable: they are like the newborn baby the Chairman has just confined to the High Authority's care.

One of the very first requirements, to my mind, is the dissemination of information. If the High Authority is able to ensure this, that in itself will be a follow-up to the Congress. Never mind whether, as I very much hope, there will be other Congresses in later years. What matters is that the High Authority should strive to see that the findings, and the openings for research emanating from those findings, should penetrate to the farthest confines of the Community, above all that they should make a direct impact on the technical experts and the economic operators.

Emphasis has been laid, for example, on the importance of conducting systematic studies for the purpose of accurately computing the incidence of maintenance costs in the sector of steel utilization. The fact that Working Party II (I think) should have put forward this point suggests in itself that the state of affairs in this regard has up to now been in all probability unsatisfactory and should be remedied forthwith.

There is a whole range of researches to be carried out, more especially concerning materials and the ways in which they are employed. Above all, there is the boundless vista of potential new demands for and demands on steel with which technology and science are called upon to deal; there is the yet more boundless vista of demands which we have to determine, since past history — past experience, if you will — teaches us that what used to be needless has now become needful, and that the fact that the human individual should be forever experiencing new needs, new inner longings, is a sure indication of steady progress and resurgent civilization.

Furthermore, the Congress faces us with a major task of scientific and technical co-ordination.

Take the complex rules on contracts and specifications, the regulations, the statutory requirements as to safety and maintenance. Safety and maintenance, of which to my mind perhaps the most important aspect — a field in which we cannot as yet tell for certain whether the battle has been won or must still go on — is fire prevention. And again that other battle, in which I personally take a special interest, as a disgruntled member of one of the most musical, but alas! the noisiest peoples in the world — the drive for noise abatement.

We have to push ahead with our investigations and researches in all kinds of fields, bearing in mind always that this is the basis for an action programme to be put in hand. We here at this august board have not actually had formulated and submitted to us a wish we have nevertheless sensed to be in the air in the course of our various encounters and exchanges today — the wish that the studies on steel utilization should go forward, that the High Authority, while in no way monopolizing or seeking to monopolize the field in regard to aesthetics or architecture, should actively promote a work which will enable all specialists on steel utilization in the building sector to obtain the data, the research results, the technical particulars they need in order to maintain a steady rate of progress and improvement in their own work.

The European Coal and Steel Community is, by decision of the six Governments, to continue as a separate entity for two years after the merger of the Community Executives. It may be, therefore, that very soon now all of us will be required to hand on the torch, as the saying is. But I may say here and now that we mean to have taken up the recommendation for the establishment, in a form to be examined by the High Authority as a body, a of Steel Utilization Study Centre, or for the taking of steps on this basis to enable Study Centres to be set up in the six countries and support for the idea to be secured in the future from those in charge of the so desired, the so essential advance of European economic integration.

We have not, Mr. Chairman, even as Members of the High Authority, any special devotion to steel or to coal as such — and a thankless and difficult devotion it would be if we had, given the problems they are constantly raising for us just at present. But we are devoted to the effort we Members are empowered to deploy through coal and steel, as bearing a share of responsibility in our own sphere for the political and economic unity of the six European States.

In conclusion, then, I feel we can point to the political significance of this Congress, to which have come speakers from so many countries of the world, artists, scientists, technicians, engineers, public servants, scholars — all those who in this troubled yet, all in all, fascinating age represent the irreplaceable fount of progress and civilization.

The political conclusion which I feel can be drawn today is that once again our European Coal and Steel Community has shown itself, as have the two sister Communities, to be an open Community, in the sense that, even if we willed it so, it would be wholly impossible to confine our work to a mere dialogue between the European Executives and the six national Governments, without rather accomplishing that harmony of voices which is today with us in tangible form, yet which sounds all the time in our ears as the background and stimulus to our labours, because we know the hopes and aspirations, the ardour and will to work that are thrilling through the six countries of our Community. And we feel that your presence has once again served to demonstrate how the most qualified opinion in Europe should act as a democratic control on our activities, should furnish a critical assessment and constant stimulus in the process of enabling the objectives of the Treaty and of our own inmost hearts to be pursued and attained to the maximum satisfaction of all.

Such, to my mind, is the meaning of this Congress.

The theme we have been set could not have been fully thrashed out in three days. The theme set the High Authority by the Congress will receive its every attention, so that yet again, and even more than heretofore, the European Coal and Steel Community shall keep step with each one of you, and so with economic and — in the highest sense of the term — political developments in the six Community countries. Yet we know that beyond those countries and alongside them stand all the peoples of the world. Among them are those peoples to whom our Chairman, M. Jeanneney, has rightly referred, who run steel industries of their own as, it may be, a gratuitous prestige symbol. But they are peoples who, being in the initial stage of their development, look to those more advanced and more industrialized than themselves not so much even for aid as for example. We want this Congress of ours and the follow-up to it, the work we have done in these days and the work we and our successors will do in the future, to furnish both aid and example. To come then, this time, to my final conclusion: here in Luxembourg we have heard the pulsing not only of the minds but of the hearts of men from lands with a long and splendid history, but at the same time with a greater responsibility than the rest, resolving to be worthy of the high duty to which they are called. My hope is that each of you in returning to his everyday avocations in his own country will remember not only what little he may have gained here in Luxembourg from the High Authority, but above all what he himself, here in Luxembourg, so spontaneously gave to it.

For in truth the duty falls first and foremost upon us; the design to be framed and the undertaking to be abided by are first and foremost the concern of us of the High Authority. And with this statement of our profound conviction I ask your permission, Mr. Chairman, to declare the proceedings of the first International Congress on Steel Utilization closed. (Applause.)

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