Nov 24th, 12:00 AM

Sheet Metal Panels for Use in Construction

Rolf Baehre

SHEET METAL PANELS FOR USE IN BUILDING CONSTRUCTION

Current research projects in Sweden

by

tekn.dr Rolf Baehre*

SUMMARY

Swedish research work in the field of thin-walled cold-formed sheet metal structural members, carried out at the Royal Institute of Technology, comprises theoretical and experimental studies of the load-bearing capacity of various types of sheet metal panels for use in building constructions in order to supply necessary information to code-drafting authorities.

*Professor at Royal Institute of Technology, School of Civil Engineering, Department of Structural Engineering and Bridge Building, Division of Steel Construction, Stockholm, and Prof. Arne Johnson, Consulting Engineers, Stockholm.

CONTENTS

1. DEVELOPMENT OF LIGHT-WEIGHT TECHNIQUES IN SWEDEN AND RESEARCH NEEDS

Light-weight techniques - in the borad sense of the term - have a long tradition in Sweden where timber has been a dominant building material. Particularly in the single-family house sector, this tradition was the starting point for prefabricated lightweight elements for walls, floors and roof structures, and the field of application, especially for partitions and external wall structures, has been broadened so as to comprise these building components even in blocks of flats.

Successive replacement of timber components by thin-walled cold-formed sheet elements in the single-family house sector was a possible line of development, but tradition, cost aspects and production apparatus geared to timber components hampered such development.

The breakthrough for cold-formed sheet products took place instead in the field of industrial building in the form of corrugated sheeting made of aluminium and galvanized steel. Development began on a modest scale at the end of the fifties and accelerated in the middle of the sixties when Swedish steelworks started production of cold-formed products suited for building purposes.

FIG 1.1 shows the consumption of corrugated sheeting for industrial roofs and walls over the period 1865-73 as an illustration of this development, and also shows what is probably the most sweeping change in market conditions ever to take place in the history of Swedish building. Market coverage in the field of industrial building is now about 90 % for roofs and about 65 % for walls. Total consumption of corrugated aluminium and steel sheeting in 1973 was approximately 20 million m^2, or about 2,5 m^2 per capita.

The areas indicated for further developments in the use of corrugated sheeting are roofs for multistorey and single-storey residential buildings and wall cladding for buildings of lightweight external walls.

Parallel with this development in the field of structural surface elements, there was increasing interest in cold-formed sheet products to perform beam functions such as purlins, floor elements and wall studs of the channel and Z type. These structural elements are now replacing, to an increasing extent, both hot-rolled steel sections and structural components of timber.

Once the construction technique had become established, attention concentrated on improvement of existing products and utilisation of new fields of application.

In the field of materials there could be noted a trend towards higher strengths and better external protection, and in the field of forming an endeavour to apply advanced forming techniques in such a way as to obtain products which were dimendionally accurate, aesthetically pleasing and suitable from the point of view of strength.

One result of this trend which may be mentioned is the new generation of corrugated sheeting with intermediate stiffeners (FIG 1.2) which permits optimum utilisation of the strength of the material, the yield strength being approximately 340 N/mm^2.

Issues concerning the mode of action and strength of the connectors also became of interest, and this gave rise to a comprehensive study of different types of joints [1] and a preliminary code relating to the analysis, design, construction and control of these joints [2].

Increased knowledge concerning the behaviour of cold-formed products at the working and ultimate stages under different types of loading had the result, however, that product development also progressed towards building systems for residential buildings in the form of prefabricated planar building components and also room components. As early as 1966, development of such a building system for multistorey buildings, project G2000 (see FIG. 1.3) was put in hand, based on cold-formed linear structural components which are connected so as to form loadbearing wall and floor elements.

The project was discontinued in 1968 owing to the decline in demand in the house building sector; however, the work produced a number of spin-off effects which were the starting points for purposeful research and development work which will be described in the following. However, the project also gave rise to views of a more general nature concerning development of lightweight building systems with regard to constructional and functional requirements, a brief review of which is given below [3], [4].

2. SOME GENERAL VIEWS OF THE DEVELOPMENT OF LIGHTWEIGHT BUILDING SYSTEMS [3], [4]

The vision of a general building system that satisfies all conceivable requirements which may be made concerning the characteristics of the final product is as old as our ability to build room-enclosing structures. One realistic aim should be to create an industrially manufactured building system which meets reasonable requirements concerning adaptability within a given framework and is, at the same time, characterised by a proper balance between function, dwelling environment and economy.

If we consider the functional requirements of the final product, the term 'building system' can be described as the <u>technical system</u>, i.e. the totality of the building components which satisfy the technical functional requirements and contain certain technical partsystems. The frame system which performs a loadbearing and stabilising function is <u>one</u> such partsystem. "Space-enclosing components" constitute another partsystem, and "supply components" a third one (see FIG. 2.1).

The form taken by the technical system, defined as the totality of the building components which mist satisfy the technical functional requirements concerning all forms of utilisation, is governed by overriding requirements concerning its characteristics, such as function, safety, functional stability, environment, economy and adaptability, see FIG. 2.2. Of these requirements, that concerning adaptability occupies a special position and should be subjected to a critical scrutiny with regard to the forms of utilisation, since this requirement exerts a dominant influence with regard to economy.

FIG. 2.2 represents a hypothesis concerning adaptability requirements and examples of optimisation criteria. Apart from production and annual costs, long-term aspects such as reliability, conversion alternatives and demolition including recovery of the materials and the problem of refuse handling, should be given special attention.

The decision base compiled on the basis of these considerations will have decisive significance concerning the construction of the subsystems and the choice of materials.

This is illustrated in FIG. 2.3 which gives examples of technical subsystems in residential buildings and the associated requirement specifications.

The selection of technical subsystems, which together represent the technical system, has been laid out in a systematic manner for the sake of clarity. In practical application, it may be necessary to combine some subsystems into one unit. Furthermore, certain subsystems can at the same time meet functional requirements relating to different subsystems.

There are two main arguments which support the idea of using sheet metal components in building systems. These are

1. <u>In choosing the materials for the structural system and in designing it, the matter of residual value with regard to demolition, conversion, material recovery and refuse handling, is part of the overall optimisation process</u>.

   Considerations of the future make the matter of value in general, and the matter of residual value in particular, of pressing interest. If this argument is accepted then a differentiated view must inevitably be taken concerning the future value of the building in relation to the basic investment, represented by the production cost.

2. A substantial proportion of the building process can be carried out using the methods and quality requirements of processing industry.

   Industrialisation of the manufacturing process mainly refers to planar elements, and the room is made up on the building site - the degree of processing is low.

   In the case of lightweight structures based on sheet metal, industrial manufacture of the basic products is essential, and processing of the product so as to form a volume element, a natural continuation of the manufacturing process. By the use of appropriate complementary materials, a high degree of finish can be accomplished in an industrial manufacturing process.

The above also implies that research efforts in the field of lightweight construction must include consideration of the requirements which the characteristics of the end product must satisfy, and also the actual manufacturing, transport and assembly problems.

Among the technical subsystems illustrated in FIG. 2.3, floors and walls which have both a loadbearing and space-enclosing function, occupy a central position with regard to development, since the requirements concerning their characteristics are complex. The following five characteristics are in this connection of particular interest since, in principle, they are governed by codes and their quality is partly determined by the wishes of the developer.

I	LOADBEARING CAPACITY	(GENERAL REQUIREMENT)
II	STABILITY AND STIFFNESS	(" ")
III	SOUND BARRIER	(FUNCTIONAL REQUIREMENT)
IV	FIRE BARRIER	(" ")
V	CLIMATIC BARRIER	(" ")

The above requirements can be met by means of material combinations. In this context, choice of appropriate components is of particular importance for the quality and economy of the structures. The principle aimed at in choosing materials and combining components should be that several requirements are satisfied at the same time.

Exemples of the general construction of floors and walls on the basis of sheet metal components are given in FIG. 2.4 and FIG. 2.5. As a complement to this, exemples of construction are given in FIG. 2.6 and FIG. 2.7.

3. THE RESEARCH PROGRAMME FOR 1971-1980 AT THE DEPARTMENT OF STEEL CONSTRUCTION, ROYAL INSTITUTE OF TECHNOLOGY, STOCKHOLM

On the basis of considerations concerning the future position of lightweight building techniques in the residential sector, and a fundamental study [5] of the applicability of thin-walled cold-formed corrugated sheeting in light-weight building systems, a proposal was made in 1969 concerning a research programme for the period 1971 - 1980. This is shown in FIG. 3.1.

The object of this project is to produce rules for analysis and design, and to elucidate the constructional and economic conditions concerning the use of thin-walled sheet panels in the building sector.

The research project is carried out mainly with the support of grants from the National Swedish Council for Building Research to research students at the Department of Steel Construction. At the same time, research commissioned by industry is also carried out if it contributes to the body of knowledge comprised in this field of research and can thus be integrated into the project.

As will be seen in FIG. 3.1, some of the subprojects are to a certain extent related to one another, which means that a research team of about 7 people is engaged on the project.

In principle, the research project comprises four different levels:

I Fundamental studies concerning the design of partial components with regard to the loads which occur in practical application.

II Optimisation studies with regard to the functional, production engineering and assembly aspects.

III Development work of an overriding character with particular attention to lightweight building systems for planar and room elements, including the building technology aspects and interaction with materials of a different type such as boards and in-situ concrete.

IV Special development work regarding building components of specific function such as room elements for dwellings, replacement systems for modernisation, and planar elements for buildings with a high flexibility requirement.

To sum up, the present position (1975) concerning this research project is as follows.

Subproject A: The action of normal force

The object of this project is to draw up design rules of practical applicability for sheet panels which are subject mainly to the action of normal force, on the basis of a numerical analytical model for sheets of any cross section and with the aid of experimental investigations.

The practical application is to be found in the field of loadbearing walls for lightweight building systems in residential and office buildings, as well as hall buildings with loadbearing external walls.

The experimental investigations comprise channel sections with variable geometrical conditions and different types of intermediate stiffeners, special consideration being given to normal manufacturing tolerances. Evaluation of the results and final analysis of the project is expected to be complete in 1975 (doctoral thesis).

Subproject B: Diaphragm action

The object of this project is to draw up design rules for structural surface elements made up of channel sections, subject mainly to shear loading, on the basis of a numerical analytical model and with the aid of experimental investigations.

The practical application is to be found in the field of "loadbearing walls with a stabilising function" and "floors and roofs with a diaphgragm function". The problem is also of significance in conjunction with the transport and erection of room units and large shear wall structures. Different truss models have been studied and experimental work is in progress on the elements comprised in the panel under varying geometrical conditions. Evaluation and final analysis of the project will be carried out in 1975 (doctoral thesis).

A special test frame hås been constructed which will also be used for standard tests on panels made up from corrugated sheets.

Within the framework of this subproject the diaphragm action [6] of corrugated sheeting has also been dealt with, with the result that a proposal for their analysis, design and construction has been adopted by the National Board of Urban Planning as a provisional code [7].

Subproject C: Transverse loading

The object of this project is to draw up design rules for structural surface elements made up of channel sections, subject mainly to transverse loading, on the basis of a numerical analytical model and with the aid of experimental investigations.

The practical application is to be found in the field "floor and roof structures". The complex of problems comprises both physiological and safety aspects in connection with the functional requirements about floor slabs. Of great interest is the effect of intermediate stiffeners (grooves) on stiffness and bearing capacity of the panels.

Theoretical investigations have been carried out in connection with subproject A and experimental investigations concerning ∏-shaped profiles with various types of geometrical properties are under progress.

Evaluation and final analysis of the project will be carried out in 1975 (doctoral thesis).

Subproject D: Composite elements

The object of this subproject is to work out design rules for surfacing elements of sheet panels with compressed upper-flange of plywood board. These rules will be based upon an analytical design model, verified by experimental studies [8].

The practical application is to be found in the field "floor and roof structures" in lightweight building systems, single-family houses and hall buildings.

Analytical models have been constructed and the results of an extensive series of experimental investigations relating to the action of bending moment and shear force are being analysed. Final treatment of the project will take place in the spring of 1975 (doctoral thesis).

Future work comprises more general treatment of composite structures consisting of sheeting and boards such as plasterboard, wood fibre boards and concrete.

Subproject E: Joints in sheet metal panels

The object of this project is to prepare design recommendations for joints in sheet metal panels which are permanently subjected to repeated loading, as a complement to Document D8:1973 of the National Swedish Council for Building Research. Joints primarily dealt with are riveted connections and non-conventional screwed connections.

A review of literature and a study of the factors which affect the capacity of the joint to resist load have been carried out and a test programme has been drawn up.

Fatigue tests are in progress and are expected to be complete in the autumn of 1975, after which evaluation and final analysis of the project will be carried out.

The project also includes preparation of draft standards for type testing of fasteners and connections, in close association with standardisation work being carried out by the Institute for Standardisation in Building.

Subproject F: Local application of load

The object of this project is to study the problem of concentrated load transmission in sheet metal structures, as a complement to the subprojects described above.

This problem is of interest in conjunction with the splicing and jointing of structural elements, and also in conjunction with local point loads.

An outline study of the problems involved has been carried out; tests necessary in conjunction with the preparation of recommendations will be performed in 1975.

The primary aim of future work is to complete subprojects now in progress. This will be done in 1975. There will remain problems in connection with the design of composite building elements subjected to the action of combined loading, such as structural surface elements of metal sheeting under the simultaneous action of different types of loading, composite structures consisting of metal sheeting and different types of materials, and the problem of stability in complex building systems.

This work will be carried out over the period 1975-77.

Parallel with these fundamental studies, the development section of the overall programme (Items I, L, L, M in the research programme) will be introduced by a review of the functional, constructional and manufacturing requirements which form the basis of a general requirement specification. This work will lay the foundations for the actual development work which is expected to commence in 1976.

4. PRESENTATION OF THE RESEARCH RESULTS

The following brief presentation of some of the results of experimental investigations relates to corrugated sheeting (4.1), cold-formed channel sections (4.2) and connections (4.3).

Investigations concerning corrugated sheeting have been completed for the present, and have resulted in modified analytical rules. Compare the AISI Standard [9], [10].

Investigations concerning cold-formed channel sections and connections are in progress. The results of these investigations and the associated theoretical analyses will be published in their entirety later on.

4.1 Corrugated sheeting

The analytical rules contained in the AISI Standard were previously used in Sweden in determining the loadbearing capacity of corrugated sheeting. Use of materials of higher strength ($F_y \approx 340$ N/mm^2) resulted in the reduction of material thickness which entailed a greater slenderness ratio in unstiffened cross sections. The result of this was that the width-thickness ratio of parts of the cross section in compression reached or exceeded the limiting values specified in the AISI Standard, and it was therefore necessary to verify these design rules experimentally.

The investigations comprised determination of the loadbearing capacity of corrugated sheeting in relation to the action of bending moment, bearing pressure and simultaneous action of bending moment and shear force.

The results of these investigations showed that application of the AISI Standard for the design of corrugated sheeting entailed in some cases an overestimation of the loadbearing capacity of the sheeting. There was a drop in the use of materials of higher strength.

This gave rise to the development of new section shapes and comprehensive testing of corrugated sheeting with intermediate stiffeners in both the flanges and the webs.

The results of these investigations are outlined in Sections 4.11-4.13.

The requirement concerning optimum utilisation of the corrugated sheeting also resulted in the stabilising capacity of the sheeting by diaphragm action becoming the focus of interest. A review of existing literature [6] and the results of previous tests indicated that the diaphragm action of corrugated sheeting with no intermediate stiffeners could be satisfactorily determined analytically. No such material was however available for sheeting with intermediate stiffeners. In order to determine the diaphragm action of such sheeting and to check the design conditions according to the Swedish regulations [7], an extensive test series was performed on corrugated sheeting Type NJA-TRP 50, 70 and 110; this is presented in outline in Section 4.14.

4.11 Bending moment

The moment capacity of corrugated sheeting is dependent on its susceptibility to buckle in the flanges and web. Under normal geometrical conditions, the share of the capacity taken by the web is 30-50 %. A reduction of the contribution provided by the web, as a result of web buckling, therefore exerts a comparatively large influence on the moment capacity. The flange, which is subjected to bending compression, often has a high slenderness ratio and exhibits a propensity to buckling which also affects the straightness of the web. When the web is buckled, Navier's principle concerning linear distribution of bending stress is only approximately valid, and the greater the depth of buckling, the less representative it becomes.

Under normal geometrical conditions, the ultimate stage is reached when the compressive stress at the junction between web and flange is equal to the yield stress - in exceptional cases, when yield occurs in the flange in tension.

A series of test specimens with variable width-thickness ratios for the flanges and webs (see FIG. 4.1) and a test rig for gravity loading at the quarter points of the specimen (see FIG. 4.2), were chosen for the experimental investigations.

The central deflection of the elements and the distribution of strain over the cross section at the centre of the span were recorded during the tests.

The test results are presented in FIG. 4.3, in which the vertical axis shows the ratio M_{exp}/M_{AISI} and the horizontal axis the fictive slenderness ratio α of the web and the ratio s/t. The presentation relates to 78 tests, the points plotted representing the means of two or three tests on geometrically similar sections.

The symbols employed in the diagram are

M_{exp} = ultimate moment obtained in the test

M_{AISI} = ultimate moment calculated according to the AISI Standard, the effective width and web buckling being taken into consideration

F_y = yield stress of the material = 340 N/mm^2

F_{bc} = buckling stress of the web, determined on the basis of the buckling factor k = 23,9 for linear stress distribution (AISI)

s = width of the web

t = thickness of the steel core determined with the galvanized coating removed

$$\alpha = \sqrt{\frac{F_y}{F_{bc}}} = \frac{s}{t} \sqrt{F_y / \frac{k \pi^2 E}{12(1-\gamma)^2}} \equiv 86 \cdot 10^{-3} \left(\frac{s}{t}\right)$$

M_{F_y} = ultimate moment calculated on the assumption that the yield stress F_y is reached at the junction between flange and web

M_{StBK} = ultimate moment according to Swedish Steel Construction Code 70

BFT = wide flange in compression

SFT = narrow flange in compression

The full line M_{AISI}/M_{F_y} plotted in the diagram represents the ratio of the ultimate moment according to AISI to F_y. This curve is obtained if, in calculating the permissible stress according to AISI, consideration is given to the factor of safety of 1,67 relating to the yield stress F_y, and also to the factor of safety of 1,23 relating to the critical buckling stress F_{bc}, which means that the phenomenon of buckling for the strength class in question, according to calculations, occurs when α = 1,16 (s/t ≈ 133). On the other hand, an upper bound is given concerning the validity of the AISI Standard, set at s/t = 150 (α ≈ 1,30)1

Examination of the test results indicates that, over the approximate range $90 \leq s/t \leq 130$ $(0,7 \leq \alpha \leq 1,15)$, the AISI Standard gives an overestimation of the loadbearing capacity of corrugated sheeting. This is obviously due to the fact that the safety requirements in force do not take into account web buckling over this range.

In the Swedish Steel Construction Code [11], this effect is considered by the provision of a transition region in the buckling curve which takes into account the effect of web buckling when $\alpha \geq 0{,}67$ ($s/t \approx 77$). This reduction is illustrated in FIG. 4.3 by the discontinuous M_{StBK}/M_{F_y}.

The above discrepancies between M_{exp} and M_{AISI} warranted further study of the distribution of strain over the web. This is given in FIG. 4.4 for slenderness ratios of s/t = 163, 122 and 82. It is seen that when the applied bending moment approaches the ultimate value, the linear distribution of strain over the web is replaced by a concave distribution which gives rise to a reduction in the capacity of the web to resist load. During the tests, an increasing depth of buckling was observed in the parts of the web in compression.

As a consequence of this observation, web buckling is taken into consideration in the Swedish Sheet Panel Code [9] by means of an analytical model (see FIG. 4.5) in which calculation of the moment capacity of the web is based on an effective width (s_{e_1}, s_{e_2}) determined in view of the buckling configuration. The values are

$$s_{e_1} = 0{,}76\ t\ \sqrt{E/F_y} \qquad\qquad s_{e_2} = 1{,}1\ t\ \sqrt{E/F_y}$$

which gives $s_{e_1} = 345\ t\sqrt{F_y}$ and $s_{e_2} = 500\ t\sqrt{F_y}$. This rule is valid for $s_o > 845\ t/\sqrt{F_y}$, and web buckling is thus taken into account if $s/t \gtrsim 90$ for $F_y = 240\ \mathrm{N/mm^2}$.

This model produces reasonable agreement with test results when the sheet thickness exceeds 0,65 mm. For extremely thin sections and pronouncedly asymmetrical sections with the compressive stress in the wide flange, however, agreement between test results and the analytical model is unsatisfactory. The explanation of this may be that buckling of the flange initiates web buckling in an unfavourable manner, and that extremely thin sections have appreciable initial imperfections.

An analytical model must therefore be found which takes into consideration these effects. It should also be mentioned that it is at present impossible to calculate the effect of web stiffeners, which, according to FIG. 4.3, increases as the sheet thickness becomes smaller and the slenderness ratio of the web increases.

4.12 Bearing pressure

The risk of web buckling is present also in conjunction with local applications of load to the sheeting, for instance at the end and intermediate supports. In the AISI Standard (3.5) this local load application R_{max} is calculated by means of an empirical relationship which takes into account the effect of the width of application of the load L_s, the sheet thickness t, the corner radius r, the section depth h and the yield stress F_y. Application of this relationship is limited to a width/thickness ratio of $s/t \leq 150$ for the web and to a maximum corner radius of $r = 4t$.

Owing to the fact that this relationship was originally produced for sections with a vertical web, there is no consideration of the inclination of the web in relation to the plane at the support. Owing to this circumstance and the fact that, in the corrugated sheeting used, there are values of $r/t > 4$ and $s/t > 150$. experimental investigations were commenced. The configuration of the test specimens is shown in FIG. 4.6. Testing comprised 78 units with a suitable variation of the above parameters. Web inclination - was varied over the range $50^{o} \leq \Theta \leq 90^{o}$ and the upper limit for the width/thickness ratio was set at $s/t \leq 170$.

It is to be noted as an important result of these tests that web inclination Θ has a considerable influence on ultimate load, and that, for $s/t \leq 170$, no significant influence on the ultimate load could be noted.

Evaluation of the test results with regard to the above parameters gave rise to the following relationship for the ultimate load at the intermediate support:

$$R_B = 1{,}8F_y \cdot t^2 \left[(2{,}8-0{,}8\cdot\frac{F_y}{340})(1-0{,}1\sqrt{\frac{r}{t}})(1+0{,}1\frac{L_s}{t})(2{,}4+(\frac{\Theta}{90})^2) \right] (N) F_y \quad (N/mm^2).$$

The values employed are

$r/t \leq 10$; $50^o \leq \Theta \leq 90^o$; $s/t \leq 170$.

There are no test results concerning the ultimate load at end supports, and it has been assumed that one half of the ultimate load applicable to the intermediate support should be a value on the safe side.

The relationship between the test results (R_{test}) and the ultimate load calculated according to the above ($R_{teor.}$) is shown in FIG. 4.7. The diagram also shows the few points which lie inside the area of validity of the AISI Standard. As will be seen, agreement with the test results is to some extent unsatisfactory.

4.13 Shear force and bending moment

At an intermediate support, the cross section of corrugated sheeting is subjected to the maximum bending moment and shear force. The above tests indicated that it was likely that the aggregate effect of these two forms of action will result in a reduction of the loadbearing capacity.

In order to elucidate the loadbearing capacity of the cross section over a support, 151 tests were carried out on sections in commercial supply which correspond to the geometrical conditions described above.

During the tests, conditions applicable to a continuous sheet over a single-span beam of span L = 0,4 L* and a line load at the centre of the span were simulated, where L* is the permissible span corresponding to the section shape in question. To make the width of loading similar to actual conditions, this was varied between 40 and 100 mm.

It was found during these tests that locally high bearing pressures in combination with web buckling due to bending stresses has an effect of reducing strength, which should be taken into consideration by means of an appropriate interaction formula.

The AISI Standard specifies that the condition

$$\left(\frac{f_b}{F_{bc}}\right)^2 + \left(\frac{f_t}{F_T}\right)^2 \leq 1 \text{ must be satisfied,}$$

where

f_b = actual bending compression in the web

F_{bc} = buckling stress of the web (see above)

f_t = actual shear stress

F_T = maximum shear stress.

For reasons given above, in the case of corrugated sheeting this criterion is not sufficient for an assessment of the loadbearing capacity. In actual fact, in the case of corrugated sheeting the contribution of $(f_t/F_T)^2$ is relatively small, while the influence of the local bearing pressure on web buckling is considerable.

This state of affairs is illustrated in the interaction diagram in FIG. 4.8, in which M_{exp}/M_{teor} is plotted along the vertical axis and R_{exp}/R_{teor} along the horizontal axis. The symbols used are

M_{exp} = ultimate moment in the test

M_{teor} = ultimate moment calculated according to Section 4.11

R_{exp} = ultimate load in the test

R_{teor} = ultimate load calculated according to Section 4.12.

For $R_{exp}/R_{teor} = 0$ the loading condition is one of "pure bending"; the test results shown correspond to the mean values of all the above ultimate tests and also the range of scatter obtained.

For $M_{exp}/M_{teor} = 0$ the loading condition is one of "bearing pressure"; the test results shown correspond to the mean values of all the above ultimate tests and also the range of scatter obtained.

The results of bearing tests have been plotted in the diagram, and also the interaction relationship which is at present laid down in the preliminary Swedish sheet metal code [9].

For reasons of presentation, only about 60 % of the test results have been plotted in FIG. 4.8; the other test results show the same tendency for the moment capacity to decrease as the local load action is increased. In the case of asymmetrical sections, the moment capacity is generally higher when the narrow flange is subjected to bending compression.

Closer analysis of the scatter in the test results indicate that section geometry, material thickness and material strength affect the level of the moment capacity. However, owing to the large number of parameters and the relatively limited number of tests applicable to each type of section, it has not been possible to give an analytical relationship of general application for the determination of the moment capacity due to the simultaneous action of bending moment and bearing pressure.

The interaction curve (full line in FIG. 4.8) given in the Swedish sheet metal code should therefore be regarded as reasonable consideration of this form of loading, the limits being "pure bending" and "bearing pressure".

It is recommended that design of shape types produced in long runs be decided by means of testing, the experimentally obtained characteristic strength being used in determining the permissible load (see Section 5.2).

4.14 Diaphragm action

The stabilising effect of corrugated sheeting has been utilised in Sweden for about the past 10 years, mainly in industrial roofs [6]. Analysis and design have been based on test results or available literature information (e.g. [12]).

In order to verify certain design rules in the provisional Swedish regulations [7] and also to have some information on the diaphragm action of corrugated sheeting with intermediate stiffeners in the flanges and web, a test series comprising 41 specimens with section types NKJ-TRP 50, 70 and 110 (as shown in FIG. 4.1) was carried out, the loading cases dealt with being "only diaphragm action", "diaphragm action incl. snow load" and "diaphragm action including wind suction".

The loading cases were simulated in a loading frame (FIG. 4.9) in which the transverse load is applied by a portal which follows the displacement movements of the test frame. The size of the field was suited to the type of section and, in the case of TRP-110, was a maximum of 1800 mm parallel to the corrugations and 4220 mm in the corrugation direction.

Interest concentrated on studies of the behaviour of the field, deformations at the ends of the corrugations and failure phenomena at end supports. For this reason the overlap splices in the corrugation direction were made excessively strong, the rivet spacing being only 100 mm. Connections between the sheeting and the supporting beam were made using a varying number of thread-cutting screws in the bottom flanges of the section. Sheet thickness was varied between 0,5 and 1,0 mm in the tests. When diaphragm force and transverse load acted simultaneously, the latter was applied to its full magnitude, after which the diaphragm force was increased from zero to maximum value.

In most tests the ultimate stage was introduced by gradually increasing lateral movements of the upper flange at the end support, followed either by web crippling at the support or pull-over failure at the screw.

It was only in the case of the smallest section depth (TRP 50) that there was in some cases global shear buckling (FIG. 4.10) of the field. The diaphragm load was however of the same order as when the above failure phenomena occurred, and the buckling phenomenon therefore has no significance in practice.

The test results for the deepest section, TRP 110, are of special interest since this has intermediate stiffeners not only in the flanges but also in the web. Owing to the slight lateral stiffness of the web, in combination with the large depth, shear deformations at the end support are relatively large. It was noted during the tests that transverse loading in combination with diaphragm action has relatively small influence on the deformations which increase slightly in the loading case "snow" and decrease slightly in the loading case "wind suction". On the contrary, however, the deformations are greatly dependent on the number of fasteners at the support.

Since the loadbearing capacity is associated with the deformations at the support, capacity can be increased by appropriate end stiffeners, which is chiefly significant in practice in cases where a structure is used primarily in diaphragm action.

FIG. 4.11 illustrates in outline the deformation behaviour of section TRP 110 of 0,65 mm nominal thickness (incl. zinc), both the deformation in the plane of diaphragm action and the lateral displacements of the upper flange at the end support being shown.

Apart from the case of the panel with special end stiffeners, failure occurred as a result of web buckling. As a rule, pull-over failure at screws only occurs in the case of sections of small depth.

As will be seen in FIG. 4.11 and FIG. 4.12, the deformations are of considerable magnitude at the ultimate stage. In the loading range of $T \approx 3\text{-}4$ kN/m which is usual in the case of normal roof structures, the horizontal displacements are generally of such magnitude that the function of the roof structure is maintained. However, flange deformations may have significance in conjunction with insulation material which, as a result of repeated deformation action, will tend to fail in fatigue or to crack.

It seems therefore that suitable deformation criteria, geared to the conditions applicable to the insulation layer, should be found as a matter of urgency as a complement to the safety requirements.

4.2 Cold-formed channel sections

Cold-formed channel sections and their variants constitute the basic element in the research programme relating to sheet metal panels in structural use such as floors and walls.

Since experimental and theoretical investigations are in progress, only a few results will be given in outline pending publication of the research reports.

4.21 Action of normal force (Researcher: P.O. Thomasson)

The object of this subproject is to ascertain the loadbearing capacity of channel shaped sheet panels subjected to an axial force, and to draw up design rules for such structural elements. These sheet panels are to be made up into wall elements which, apart from their loadbearing function, also perform a space-enclosing function (see FIG. 2.7).

Mode of action and experience gained in tests

A basic section type A, with dimensions as shown in FIG. 4.13, was chosen for the investigation. Two variants were later created on the basis of this: type B with one stiffener in the wide flange, and type C with two stiffeners.

If a panel of type A is subjected to an axial force at its centroid and it is assumed that the panel was initially straight, then it will remain straight up to a certain load which is equivalent to the critical buckling stress (local buckling). When this load is exceeded, the centroid of the effective cross section is displaced towards the narrower flange. If it is further assumed that the position of the line of action of the force does not change, then the effective cross section will be acted upon by an eccentric force. As a result, the panel will deflect and there will be a further increment to the bending moment. The growth in deflection as the load is increased will occur at an ever faster pace, due to the incremental moment and the fact that the effective bending stiffness drops as the load increases. This effect is illustrated by curve A in FIG. 4.13 and FIG. 4.14. The mode of action described assumes that there is a postbuckling region for the sheets comprised in the cross section. The final cause of the exhaustion of the loadbearing capacity is dependent on the shape of the cross section, sheet thickness, length and material properties. In practice, the principal cause of failure is an interaction between local buckling and buckling, characterised either by the strength of the material being exceeded or by a local buckling phenomenon changing the stiffness of the section in such a way that global instability occurs. The latter gave rise to failure in type C in FIG. 4.14.

By providing the wide flange with an intermediate stiffener (types D and C in FIG. 4.13), the stiffness properties and loadbearing capacity can be improved. This is evident from the curves B and C in FIGs. 4.13 and 4.14. Owing to the improved stiffness properties of the panel as a whole and the increased loadbearing capacity, however, there is an increased risk of lateral buckling of the narrow flange. This risk is particularly pronounced if, as in case C in FIG. 4.14, the narrow flange is an unstiffened element. Lateral buckling in the narrow flange takes place at wavelengths that are 5-10 times the wavelengths of the buckling waves in the wide flange. If the wide flange is provided with stiffeners, then this ratio becomes smaller. In contrast to the case of a plate supported along four sides, this type of local instability has no postbuckling region, and it is therefore of particular interest. In the tests the flanges were connected by 3 x 20 mm straps at 300 mm centres in order to prevent this type of failure. According to FIG. 4.14, however, an antimetric buckling of the flanges occurred in case C despite this bracing. The moment of inertia of the intermediate stiffeners has also been varied in the tests, but no unambiguous results have been obtained as a result. The trend of the test results shows that deflection decreases as the stiffness increases, while the differences in loadbearing capacity are sometimes affected to an insignificant extent.

4.22 Action of transverse load (Researcher: J. König)

The channel shaped panels used in this research programme are intended for use as elements in roofs and floors. A series of 31 specimens is being tested at present. The principal parameters being varied are the number of intermediate stiffeners (none, one, two) in the flange in compression.

The purpose of this series of tests is to investigate the effect of these intermediate stiffeners on ultimate load and deformations. The objects of the investigation are

- determination of the minimum stiffness of the stiffener which ensures that failure occurs by local buckling
- determination of the limiting effectiveness of the stiffeners (i.e. the buckling load for the whole upper flange) and the stiffness of the stiffeners which implies the maximum limiting effectiveness for a certain flange
- determination of that stiffener stiffness which cuts local deformations to the minimum in relation to the buckling of the whole flange and flange curling due to curvature of the beam
- determination of an economic stiffener stiffness which gives the beam as a whole the largest loadbearing capacity and the smallest deformations in relation to the material used.

Since the yield stress of the sheeting, 320-400 N/mm^2, is very much higher than was usual previously, it is of interest to find whether the higher steel grade results in an increase in ultimate values to the same extent.

In choosing the section geometry it is assumed that all beams should have the same upper flange width, 300 mm, which suits the modular system used in Sweden

(FIG. 4.15). In order to make testing procedure easier, the bottom flanges are bent outwards instead of inwards. This has no effect on the results, however. Two sheet thicknesses were selected. One series of 14 specimens which are 3,0 m long were made of sheeting 0,7 mm thick, while the second series of 14 specimens of 6,0 m length are made of 1,5 mm sheeting. This is equivalent to w/t ratios of approx. 430 and 200.

Since the aim of the test series is to investigate the failure and deformation behaviour of only the flanges in compression, the depth of the web was to be limited in order to prevent web buckling affecting the flange.

The web depths were therefore made 60 mm in the case of the short specimens and 100 mm in the case of the long ones. The widths of the bottom flanges were such that the yield stress would not be reached in the bottom flanges.

The specimens were made with the upper flanges both stiffened and unstiffened, the stiffeners being in the form of one V-shaped groove in the centre of the flange, or two V-shaped grooves at the third points of the flange. The moments of inertia of these stiffeners were made to correspond to the minimum moment of inertia calculated according to the AISI Standard, half this value and twice this value.

All specimens are tested as beams on two supports. Continuous loading is replaced by 4 point loads at a spacing of L/4.

Load is applied by means of weights which are suspended from the bottom flanges an yokes. In this way, failure at the points of application of the loads, due to web drippling, is prevented. At the same time, the upper flanges are unobstructed so that their displacement in relation to the web can be measured (FIG. 4.16). This is done by means of a movable dial gauge which is placed on the beam at the corners between web and upper flange. Since creep deformations increase as loads become higher, the load is maintained constant for five minutes before the next loading step. The load which the beam can resist for at least five minutes before failure occurs is regarded as the ultimate load.

4.23 Action of in-plane loading (Researcher: G. Nyberg)

Wall structures made up of thin-walled panels of channel section (see FIG 4.17) have, in addition to their primary duty of resisting axial and transverse loads, also a secondary loadbearing function whereby, by means of diaphragm action, they provide the requisite stability, mainly in conjunction with wind loads.

In the case of floor structures made up of channel sections also, the capacity to resist in-plane forces is of importance.

The aim of this project is to assemble data by means of a limited number of experiments so that an assessment may be made of the extent of which diaphragm action can be taken into consideration in structures of this type. A numerical analytical model will be constructed as a second stage. Together with experimental material, the analytical model will form the basis for design rules.

The shear buckling load of thin-walled unstiffened channel shaped panels, as calculated by the elastic theory, is very low. It is however to be expected that shear loads in excess of the shear buckling load according to the elastic theory can in certain circumstances be resisted by utilisation of a "supercritical region", i.e. the formation of a tension field.

In order that a tension field may be set up, it is essential for the connections between the channel-shaped panels to be sufficiently strong and to be placed in such a way that the tensile forces in the tension field can be transmitted between the webs of the adjacent panels.

The tension field formed must also be anchored in appropriately designed edge beams.

In the test series now in progress panels of widths B = 300 and 600 mm and thicknesses t = 0,7 and 1,2 mm (see FIG. 4.17) are being tested. The wall contains 14 panels of 300 mm width or 7 panels of 600 mm width. Connections between the panels and between the panels and the edge beams are in the form of spot welds.

The positions (d) of the connections in relation to the web are to be varied so as to obtain information concerning the way this affects the stiffness of the wall and its loadbearing capacity.

In a possible analytical model (see FIG. 4.18) the webs of the channel-shaped panels can be replaced by crossing members in tension and compression. When the critical shear buckling load is reached, increase in load is resisted only by the members in tension. The flanges of the panels, together with part of the web, form beams which make up a lattice in conjunction with the members in tension in the web. This can be analysed in a computer. By e.g. varying the areas of the members in tension, the load-displacement characteristics of different types of connectors can be simulated.

4.3 Connectors (Researcher: B. Nissfolk)

Design of joints in sheet metal panels takes place at present according to provisional regulations [2] based on tests to failure carried out on suitable connectors such as high-strength friction bolts, thread-forming screws, hollow rivets and spot welding (resistance welding and fusion welding)[1].

The fundamental design philosophy has been to limit the deformations of these connectors under working load, which means that the characteristics ultimate load has formally been reduced to a fictitious yield load that is characterise by progressive deformation. Owing to the lack of experience data, relatively conservative values of the permissible load have been laid down. In practice, such rigid association between safety requirements and functional requirements is often unnecessary and uneconomical.

It is however essential in order that a more liberal design method should be employed that a uniform test and evaluation method is applied with reproducible results. Another essential condition is that the deformation and strength conditions under repeated loading are also known.

In order to elucidate these conditions, supplementary testing and standardisation work is in progress concerning the test method to be applied to connections under the action of static shear force (4.31) and static pull-out force (4.32), as well as connections subjected to dynamic loading (4.33).

4.31 Shear tests (static loading)

The primary aim of these shear tests has been to determine the characteristic stiffness of different types of connections on being loaded to failure. The deformations of the connections are of primary interest in assessing the deformations in panels of e.g. corrugated sheeting, where these deformations have an evident influence on the overall stiffness.

As an example of the behaviour of the riveted connection, FIG. 4.19 shows a load-displacement diagram where load was removed and re-applied at different load levels. Two different types of test specimens are considered.

The elastic modulus of the material was separately determined as E_0 = 195.300 N/mm^2; the relatively low value indicates a certain degree of anisotropy in the hot-dip galvanized sheet.

As will be seen from the load-displacement diagram, the fictive elastic modulus (E^*) of the connection, up to about 70 % of the ultimate load, is of the order $E^* \approx 0{,}06\text{-}0{,}07\ E_0$. Expressed as the displacement factor c (mm/kN), the 'slip' of the connection is characterised by the value $c \approx 0{,}17\text{-}0{,}20$ mm/kN.

The load-deformation diagram in FIG. 4.19 also shows that deformation is to some extent dependent of the configuration of the test speciment, but that most of the displacement is attributable to the local stress concentration at the rivet. When load is removed, there is at relatively low loads a permanent deformation due to local plastic flow in the barrel of the rivet and the edge of the hole, and incomplete filling of the hole.

It is also probable that there is an increase in strength due to local strain hardening at the edge of the hole, since on removing the load from a certain load level and renewed application of the load there is a practically linear relationship between load and deformation, and a considerable reduction in the 'slip' of the connection. For the connection referred to, when the load is removed from about 60 % of the ultimate load, the residual deformation is of the order of 0,3 mm.

As is usual in joints where thin sheets are laid against one another, failure occurs due to tearing of the sheet and pull-out, or fracture of the rivet.

4.32 Pull-out tests

As will also have been seen from the in-plane tests described above (4.14), when the corrugated sheeting is utilised as roof sheeting there is a force at right angles to the plane of the connection, and this can result in pull-out or pull-over failure at the point of attachment of the bolt. In this case also, there is both a safety problem and a deformation problem.

Some test series were performed in order to elucidate the deformation behaviour and to determine the pull-over load for the types of screws, bolts and sections in question.

In an introductory test series, the dependence of the pull-over load on the section geometry and the length of the test specimen was investigated. As will be seen from FIG. 4.20, the pull-over load is dependent on both the section shape and the length of the specimen. This state of affairs resulted in a demand for standardisation of the test method (see Section 5.2) in order that reproducible results may be obtained.

In a test series comprising 50 test specimens of corrugated sheeting types TRP 50, 70 and 110, with nominal thicknesses ranging between 0,65 and 0,9 mm, the characteristic loadbearing capacity X, which applies with a probability of 95 % at a confidence level of 75 %, was then determined. It was found that

$$X = \bar{x}\ (1 - c_5 \cdot \delta)$$

where $\bar{x}$ = mean value of the ultimate load

c_5 = a coefficient dependent on the number of readings

δ = coefficient of variation.

The test results are summarised in FIG. 4.21 which also gives the characteristic loadbearing capacity. The following relationship holds approximately for all sections with 1 screw or bolt at the bottom of the section

$$X \approx 8.750 \cdot t_k^{1,72} \quad (\mathrm{kN})$$

where t_k = thickness of the core of the sheet (mm).

Somewhat variable results were obtained for sections with two screws or bolts at the bottom of the section. The following holds with good approximation per screw or bolt for section TRP 50

$$X_2 \approx 8.200 \cdot t_k^{1,65} \quad (\mathrm{kN})$$

which is thus a reduction of the pull-over load compared with the case when there is only one screw or bolt.

At the permissible load level, the deformations abount to about 6 mm, with a residual deformation of about 3 mm, which is considered acceptable in view of the fact that the load level is equivalent to the wind intensity that occurs once per 50 years.

4.33 Shear tests (dynamic loading)

There are at present no design rules in Sweden for connections in sheet metal panels which are subjected to dynamic or repeated loading. However, owing to utilisation of the diaphragm action in buildings with lightweight cranes, unintentional dynamic action in structures prone to oscillation, and wider use of sheet metal structures even outside the actual building sector, it is necessary that it should be possible to assess the dynamic strength of the connections.

This is the background to dynamic loading tests in shear connections comprising hollow rivets of steel, monel metal and aluminium.

Overall, the tests comprise 20 test series with about 20 specimens in each series. With the object of drawing up Wöhler curves, tests are performed at three different stress horizons approximately equivalent to load alternations of 30,000, 100,000 and 500,000.

Testing is performed in an MTS Universal Testing Machine in the form of tensile pulsation.

The following parameters are varied:

Stress ratio	$R = P_{min}/P_{max} = 0,1;\ 0,25;\ 0,5$
Frequency	$f = 25;\ 5;\ 1$ Hz
Sheet thickness	$t = 0,5;\ 0,6;\ 0,7;\ 0,9;\ 1,2;\ 1,5;\ 2,0$ mm
Material strength	$F_y = 250;\ 280;\ 350\ N/mm^2$
Rivet diameter	$d = 3,2;\ 4,0;\ 4,8;\ 6,4$ mm.

The specimens are loaded from 0 to P_{max}, after which load is removed to the level $\frac{1}{2}(P_{max} + P_{min})$. The dynamic load is then re-applied.

Deformations are measured with 2 extensometers, each with 4 sensors.

FIG. 4.22 shows Wöhler curves obtained from the results of these tests for hollow riveted connections with rivets of steel, monel metal, and aluminium ($d = 4,8$ mm) in 0,9 mm steel sheeting with a strength of $F_y = 350\ N/mm^2$.

As a rule, the cause of failure is yield in bearing in combination with cracking at the side of the hole and inclination of the rivet.

The curves represent a survival probability of 95 %, in agreement with the definition of characteristic loadbearing capacity according to Swedish Standards.

For steel rivets, Wöhler curves are given for stress ratios of R = 0,1; 0,25 and 0,5 (B, E, F), for aluminium and monel metal rivets for R = 0,1 (C, D). It is seen that, when plotted to double logarithmic scale, the Wöhler curves are approximately straight lines in the region of load alternations of $10^4 \lesssim N \lesssim 10^6$.

5. APPLICATION OF THE RESEARCH RESULTS

The research work described above occupies a central position within an expansive development area. This means that long-term research is often overtaken by acute needs for information which must be met, either to promote progress or to avoid mistakes. The consequence of this is an intimate interaction between research and application, but also valuable knowledge on which future research can be based, even if the individual researcher often regards interference with planned research as an irritation.

The results of experimental and theoretical investigations have found application in structural codes [2, 7, 9] and also standards documents.

Work on standardisation is performed by a technical committee "Steel structures" (TK 83) within the INSTITUTE FOR STANDARDISATION IN BUILDING. In view of the fact that in developing new products and also in conjunction with complicated structural elements, design by testing instead of analytical treatment is often resorted to, interest primarily concentrated on standardisation of testing and evaluation methods.

The following is a brief outline of standardisation projects regarding test procedures in the sheet metal sector, which are either in the course of preparation or have been completed.

5.1 Testing of corrugated sheeting

Testing specifications for the determination of the loadbearing capacity relate to uniformly distributed loading [13] and concentrated loading [14]. The standard is expected to come into force in the autumn of 1975.

5.11 Uniformly distributed loading [13]

This standard describes test methods to provide information concerning stiffness, moment capacity and support reaction in conjunction with uniformly distributed loading.

The testing equipment for determination of the span moment, support moment and support reaction is illustrated in FIG. 5.1.

Testing of continuous beams on three supports with 4 line loads in each span is permitted as an alternative test method for determination of the moment capacity over a support.

In evaluating the test results, a correction is applied with respect to deviations from nominal strength F_g and sheet thickness t according to established methods, after which the characteristic strength R_o is determined from the formula

$$R_o = R_m (1 - c_5 \cdot \delta)$$

where R_m = mean value of test results

c_5 = a coefficient dependent on the number of readings as set out in the table below

δ = coefficient of variation which is determined on the basis of a curve fitted to the test readings ($R = f(t)$)

No of readings	3	4	5	6	8	10	15	20	40
c_5	3,2	2,7	2,5	2,3	2,2	2,1	2,0	1,9	1,8

It is assumed in this connection that in the case of readings of normal statistical distribution, there is 95 % probability that the characteristic loadbearing capacity will be maintained at the 75 % confidence level.

5.12 Concentrated loading [14]

This standard dscribes the method for determination of the ability of the sheeting to support the weight of persons and temporary local loads in conjunction with insulation and roofing work.

The test procedure is described in FIG. 5.2.

Load is applied at the most critical of positions 1 and 2 by means of a wooden cube with sides of 100 mm and a height of 45 mm. The load is applied in steps, load being removed to zero between each step, and a check is made to see that the residual deflection of the loaded top of the section compared with the one immediately adjacent is not more than 3 mm.

The criterion that the sheeting must support the weight of persons is that this residual deflection must not be exceeded and that there must be no permanent depressions when load is removed from the level 1 kN. The above criterion is based on field studies in which the subjective judgment of people concerning the weight of persons was quantified in terms of the above deformations.

5.2 Testing of connections

Testing specifications for connections in sheet metal panels relate to shear tests [15] and pull-out tests [16]. A draft standard is at present under discussion, and the standard is expected to come into force in 1976.

5.21 Shear test [15]

The proposal for a test method relates to attachment of sheet metal to another sheet or to a support of metal, concrete or timber.

Alternative test rigs are shown in FIG. 5.3 for connections between thin sheets (A) and for attachment of thin sheets to thicker sheeting (B) or to supports of timber or concrete. Connections with 2 fasteners are used in the test. The width of the sheet shall be 60 mm.

The force-deformation curve is recorded during the test, the elongation being measured over the gauge length L_o as shown in FIG. 5.3.

Statistical methods are again applied in evaluating the results, the design ultimate load R_o being determined from the formula

$$R_o = R_m - k \cdot s$$

where R_m = mean value of test readings

k = factor dependent on the number of test readings (see table in 5.11)

s = standard deviation

5.22 Pull-out tests [16]

This standard relates to determination of pull-out or pull-over load at the attachment between corrugated sheeting and between sheets and supports.

Alternative test rigs for pull-out tests are shown in FIG. 5.4, the stipulation being that the span of the sample must be equal to 6 times the width of the bottom of the section. The reason for specification of this test length is that formation of a funnel around the fastener should not be prevented. On the other hand, care must also be taken to prevent the section as a whole folding up.

Load is to be applied in at least five steps, there being a return to the unloaded state between each loading step. The rate of testing must not exceed 20 mm/min.

During the test, the force-deformation curve is recorded with respect to the movement between the top of the fastener and the top of the section.

Evaluation of the results is to be performed according to the same principles as in 5.21.

6. SUMMARY

This paper is an outline presentation of research work in progress at the Department of Steel Construction, Royal Institute of Technology, Stockholm. The basis of this work is a long-term research programme relating to sheet metal panels in building construction, and also the duty of supplying the necessary information to code-drafting authorities and the business sector, in order to secure and promote the expansive development in the field of sheet metal.

Research results are reported either in separate publications or in conjunction with specifications and standards documents.

REFERENCES

[1] Baehre, R., and Berggren, L., "Hopfogning av tunnväggiga stål- och aluminiumkonstruktioner 2" (Jointing of thin-walled steel and aluminum structures), National Swedish Building Research, Report R30:1971. Distribution by Svensk Byggtjänst, Box 1403, S - 111 84 Stockholm, Sweden. (Applies for all publications from the National Swedish Building Research, below entitled BFR ...).

[2] Baehre, R., and Berggren, L., "Joints in sheet metal panels", BFR-Document D8:1973.

[3] Baehre, R., "Building Systems in Housing Construction - Points of Criticism". Contribution to the Polish-Swedish Symposium 1973, published in "Perspectives of Development of Housing Industrialization", BFR.

[4] Baehre, R., "Lightweight Structural Components for Floors and Walls". Contribution to the Swedish-Polish Symposium 1975, BFR.

[5] Baehre, R., and Thomasson, P., "Plåtpaneler i byggnadsteknisk användning" (Sheet metal panels in building construction). BFR-Report R10:1971.

[6] Baehre, R., and Nyberg, G., "Stabilisering av byggnader genom ytbärverk av profilerad plåt" (Stabilization of buildings by structural cladding of corrugated sheeting). BFR-Report R4:1974.

[7] Baehre, R., and König, J., "Skivverkan av trapetsprofilerad plåt" (Swedish Specification for "Stressed Skin Design"). BFR-Report B1:1975.

[8] Larssen, J.F., "Lättbärverk med samverkande blandkomponenter" (Lightweight structural elements comprising interacting mixed components). BFR-Report R75-1973.

[9] Tunnplåtskonstruktioner, Godkännanderegler nr 3.
Swedish Specification for design of thin-walled cold formed structural members. The national Swedish Board of Urban Planning.

[10] Specification for the Design of Cold-formed Steel Structural Members", 1968 Edition. American Iron and Steel Institute, New York.

[11] STÅLBYGGNADSNORM 70, StBK-N1 (Swedish Specification for the design of Steel structures), 1970.

[12] Bryan, E., "The Stressed Skin Design of Steel Buildings". Constrado Monographs, Crosby Lockwood Staples, London 1972.

[13] Profilerad plåt. Bestämning av bärförmåga vid jämnt fördelad last. Förslag till Svensk Standard SIS .. .. ..
(Corrugated sheets. Load capacity under distributed transverse loading. Draft Swedish Standard).

[14] Profilerad plåt. Bestämning av bärförmåga vid punktlaster. Förslag till Svensk Standard SIS .. .. ..
(Corrugated sheets. Load capacity under local transverse loading. Draft Swedish Standard).

[15] Tunnplåtsförband. Bestämning av skjuvhållfasthet. Förslag till Svensk Standard SIS .. .. ..
(Joints in thin-walled structural members. Shear capacity. Draft Swedish Standard).

[16] Infästning av profilerad plåt. Bestämning av hållfasthet vinkelrät plåtplanet. Förslag till Svensk Standard SIS .. .. ..
(Fastening of corrugated sheets. Load capacity when subjected to forces normal to the plane of sheeting. Draft Swedish Standard).

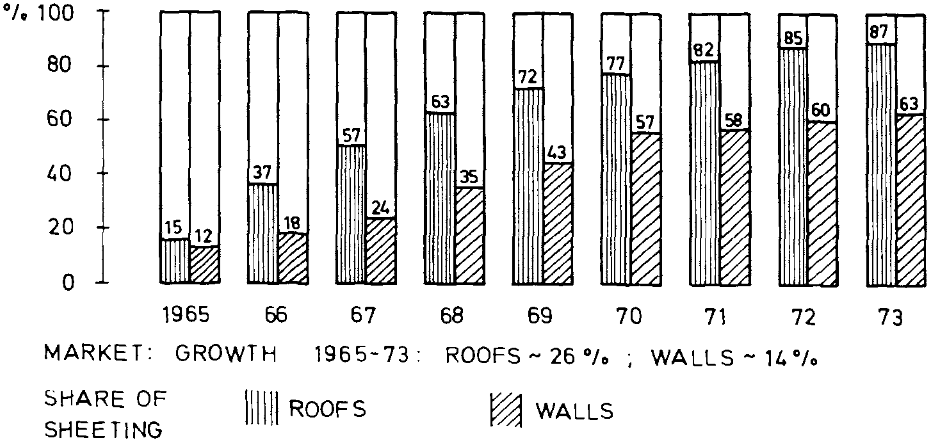


FIG. 1.1 Growth of the market for corrugated sheeting for industrial buildings over the period 1965-73, in relation to the total market

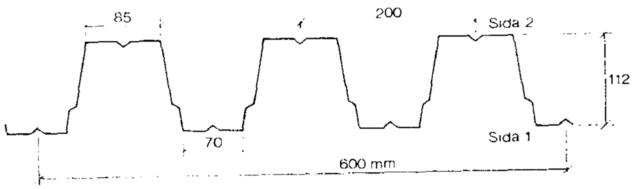


FIG. 1.2 Examples of corrugated sections with intermediate stiffeners (type NJA-TRP 110)

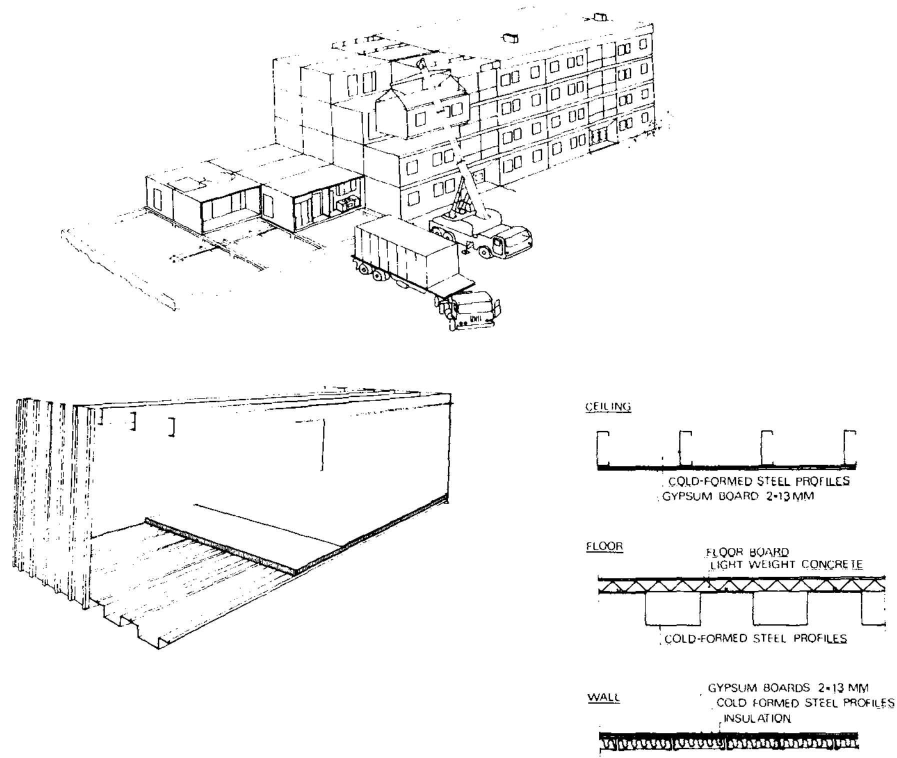


FIG. 1.3 Room unit with sheet metal components for building system used in residential building (type G 2000)

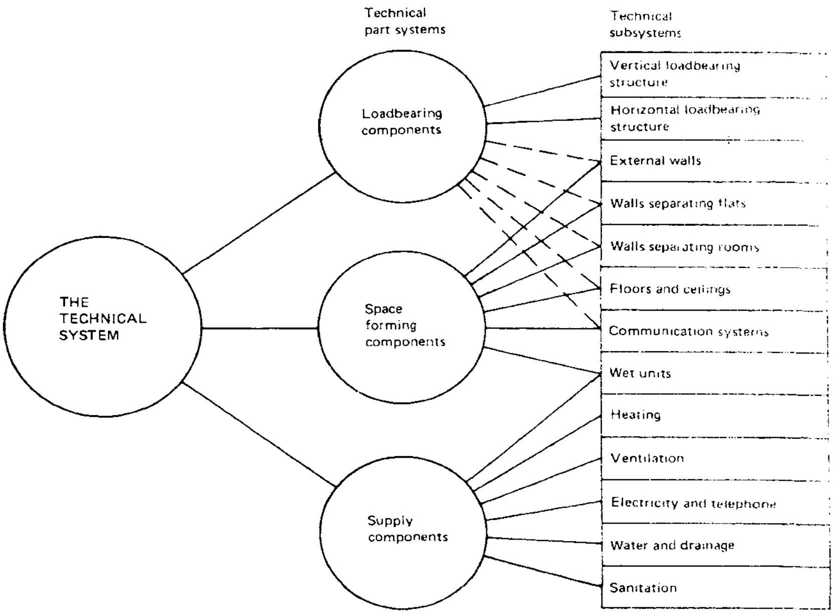
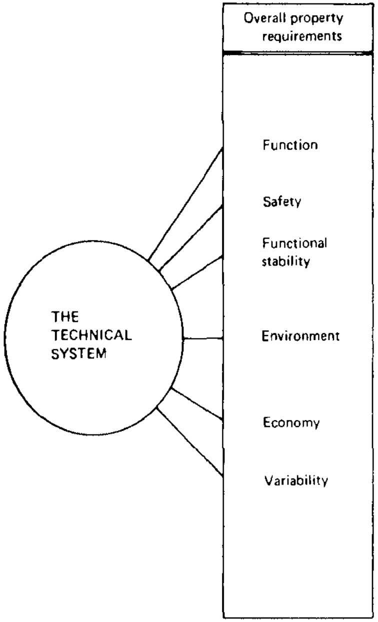


FIG. 2.1 Technical system - part systems - subsystems



Technical part systems	Overall functional requirements	Variability requirements	Optimisation criteria
Loadbearing components	Safety against collapse under the action of external load and functional stability over the intended life	No requirement during the intended life	1) Suboptimization of structure 2) Demolition position 3) Life 4) Recovery of materials 5) Refuse disposal
Space enclosing components	Internal and external environmental requirements and protection requirements with regard to climate, sound, fire . .	Moderate requirements regarding variability of space enclosing components in order to bring about a change in the layout	1) Production cost 2) Annual costs; heating, cleaning, maintenance 3) Conversion 4) Recovery of materials 5) Refuse disposal
Supply engineering components	Hygienic and physiological comfort requirements; capacity requirements	Replaceability with regard to life, energy problems, technical progress in this field	1) Primary investments 2) Annual costs 3) Reliability 4) Energy costs 5) Replaceability

FIG. 2.2 Requirement specifications for the technical system in residential buildings

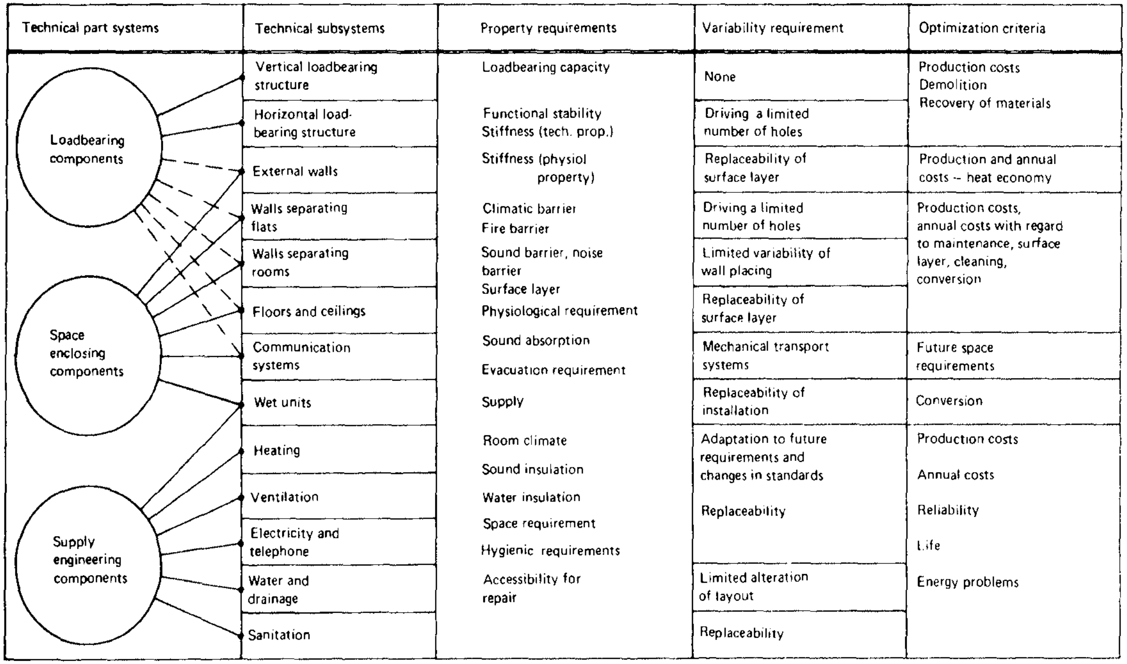


FIG. 2.3 Examples of technical subsystems and associated requirement specifications

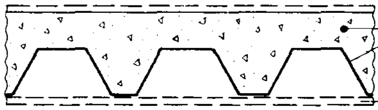
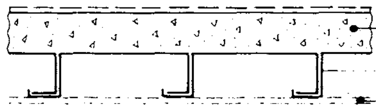
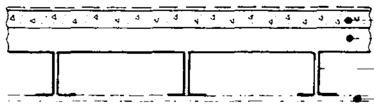
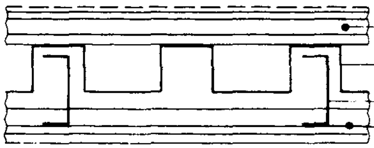
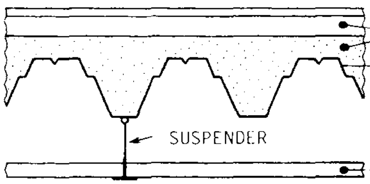
TYPE	FORM OF CONSTRUCTION	TYPICAL CONSTRUCTION	FUNCTION
A	CORRUGATED SHEETING + CONCRETE (WITH OR WITHOUT COMPOSITE ACTION). IF NECESSARY: FIRE PROTECTION BY FALSE CEILING		I,II,III IV
B	SHEETING STRUCTURE WITH CONCRETE SLAB ON TOP (IF NECESSARY: FIRE PROTECTION BY FALSE CEILING)		II,III,IV I,II IV
C	SHEETING STRUCTURE WITH INSULATION LAYER AND CONCRETE SLAB ON TOP (IF NECESSARY: FIRE PROTECTION BY MEANS OF FALSE CEILING)		IV III (V) I,II IV
D	MULTILAYER SHEETING STRUCTURE OF DRY CONSTRUCTION CONSISTING OF BOARD MATERIAL		III,IV I,II I,II III,IV,V
E	SHEETING STRUCTURE WITH FALSE CEILING SUSPENDED IN A FLEXURALLY WEAK MANNER		III,IV I,II III,IV,V
F	LIGHTWEIGHT SHEETING FLOOR OF DRY CONSTRUCTION CONSISTING OF BOARD MATERIAL (IF NECESSARY: FIRE PROTECTION AND CLIMATIC PROTECTION)		III (IV,V) I,II (IV,V)

FIG. 2.4 General construction of floor slab using sheet metal components

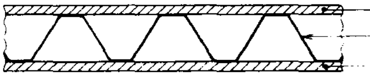
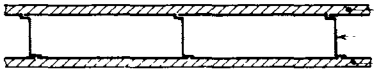
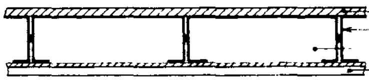
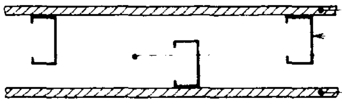
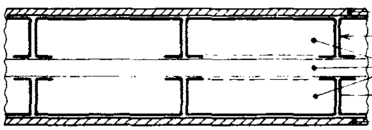
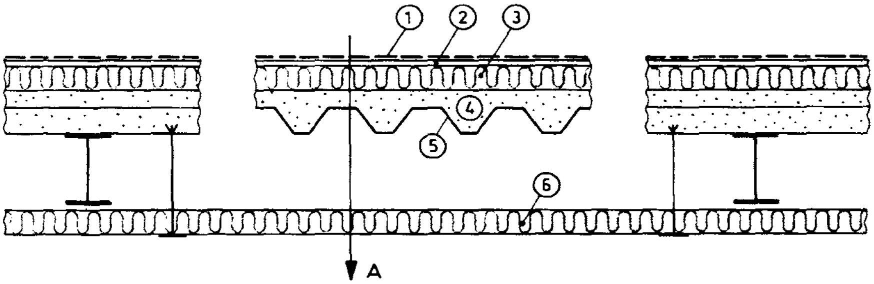
TYPE	FORM OF CONSTRUCTION	TYPICAL CONSTRUCTION	FUNCTION
A	CORRUGATED SHEETING WITH EXTERNAL LAYER OF BOARD MATERIAL, WITH OR WITHOUT COMPOSITE ACTION (IF NECESSARY, FIRE PROTECTION)		III IV I II III IV
B	CLOSED SHEETING CASSETTES WITH EXTERNAL LAYER OF BOARD MATERIAL, WITH OR WITHOUT COMPOSITE ACTION (IF NECESSARY: FIRE PROTECTION)		III(IV) I II III(IV)
C	SHEETING PANELS WITH EXTERNAL LAYER OF BOARD MATERIAL, WITH OR WITHOUT COMPOSITE ACTION. (IF NECESSARY: CLIMATIC PROTECTION AND FIRE PROTECTION)		III IV I II (IV)(V) III IV
D	SANDWICH UNIT WITH LIGHTWEIGHT CORE MATERIAL AND EXTERNAL LAYER OF BOARD MATERIAL (IF NECESSARY: FIRE PROTECTION)		I II III V I (IV)
E	DOUBLE-SKIN LIGHTWEIGHT WALL OF THIN-WALLED COLD-FORMED SHEETING SECTIONS AND EXTERNAL LAYERS OF BOARD MATERIAL. (IF NECESSARY: ADDITIONAL SOUND INSULATION AND CLIMATIC PROTECTION)		III IV I II (V)(III) III IV
F	DOUBLE-SKIN LIGHTWEIGHT WALL OF SHEETING PANELS WITH EXTERNAL LAYERS OF BOARD MATERIAL AND ADDITIONAL PROTECTION MEASURES (CLIMATE, SOUND, FIRE)		IV I II III IV V I II IV

FIG. 2.5 General construction of walls using sheet metal components



1 FLOOR COVERING

2 19mm PLYWOOD SHEET

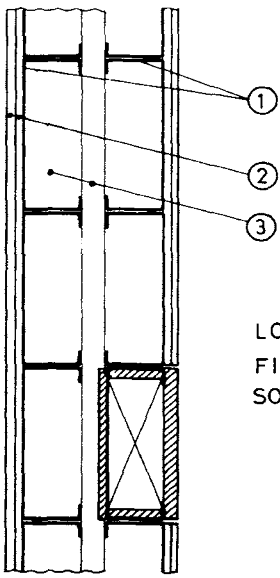
3 50mm MINERAL WOOL SHEET

4 SAND FILL 100 kg/m^2

5 CORRUGATED SHEET

6 SOUND INSULATING FALSE CEILING (TYPE RW 359)

FIG. 2.6 Example of floor using sheet metal components



1 COLD FORMED CHANNEL SECTIONS 300×100×1,5 (SPOT WELDED)

2 2×13mm PLASTERBOARD

3 MINERAL WOOL INSULATION (100mm + 50mm)

LOADBEARING CAPACITY APPROX. 15 tons/m
FIRE PROTECTION > A60
SOUND INSULATION $\bar{R} \approx 57$ dB

FIG. 2.7 Example of double wall using sheet metal components

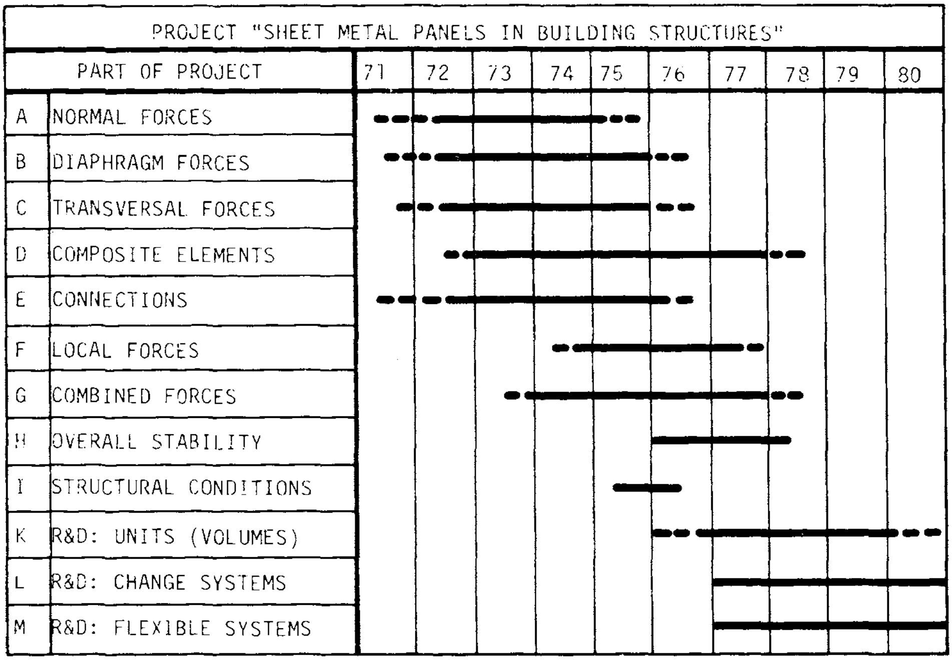


FIG. 3.1 Research programme for the period 1971-1980 at the Department of Steel Construction, Royal Institute of Technology, Stockholm

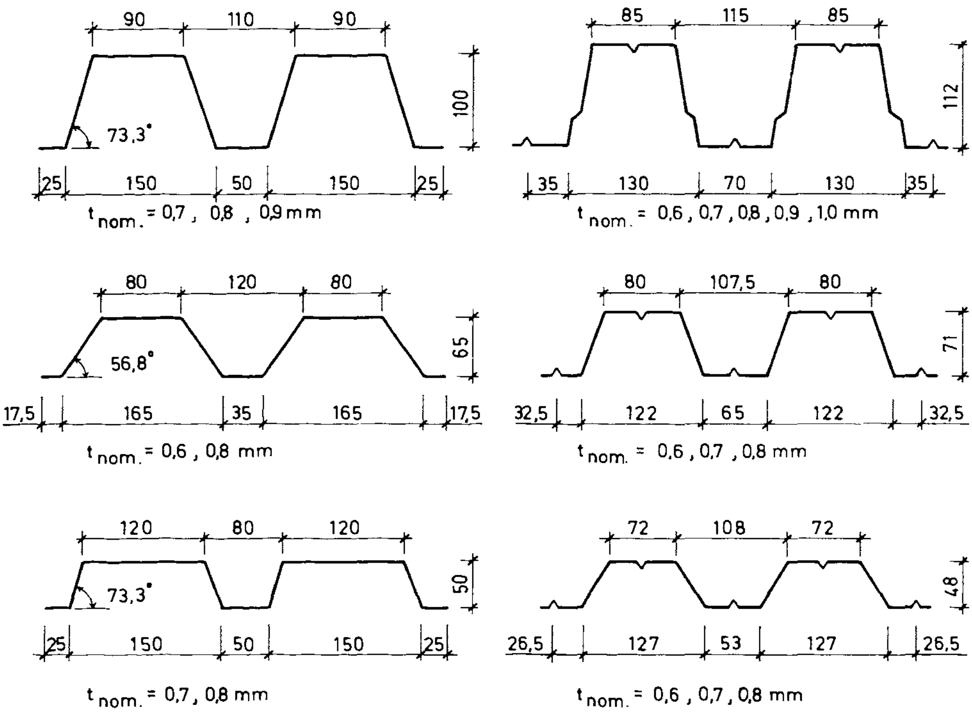


FIG. 4.1 Test specimens for bending moment and support tests

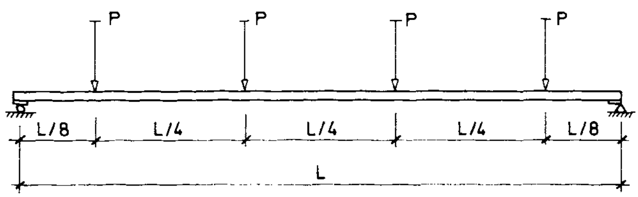


FIG. 4.2 Test rig for bending moment tests

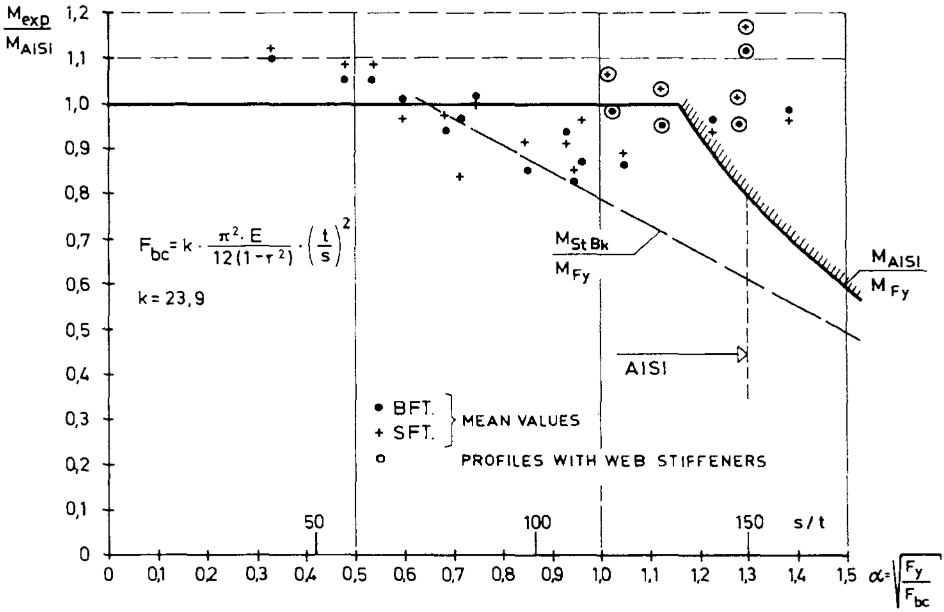


FIG. 4.3 Results of 78 tests on the action of bending moment in relation to the loadbearing capacity according to AISI

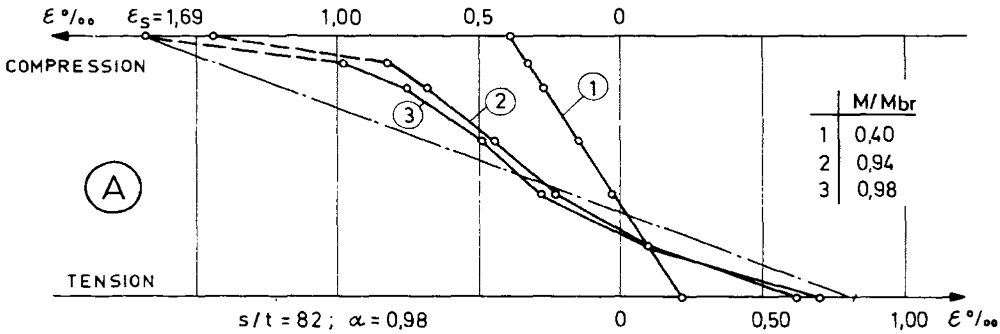


FIG. 4.4 Example of strain distribution measured over the cross section of the web (M_{br} = measured ultimate moment)

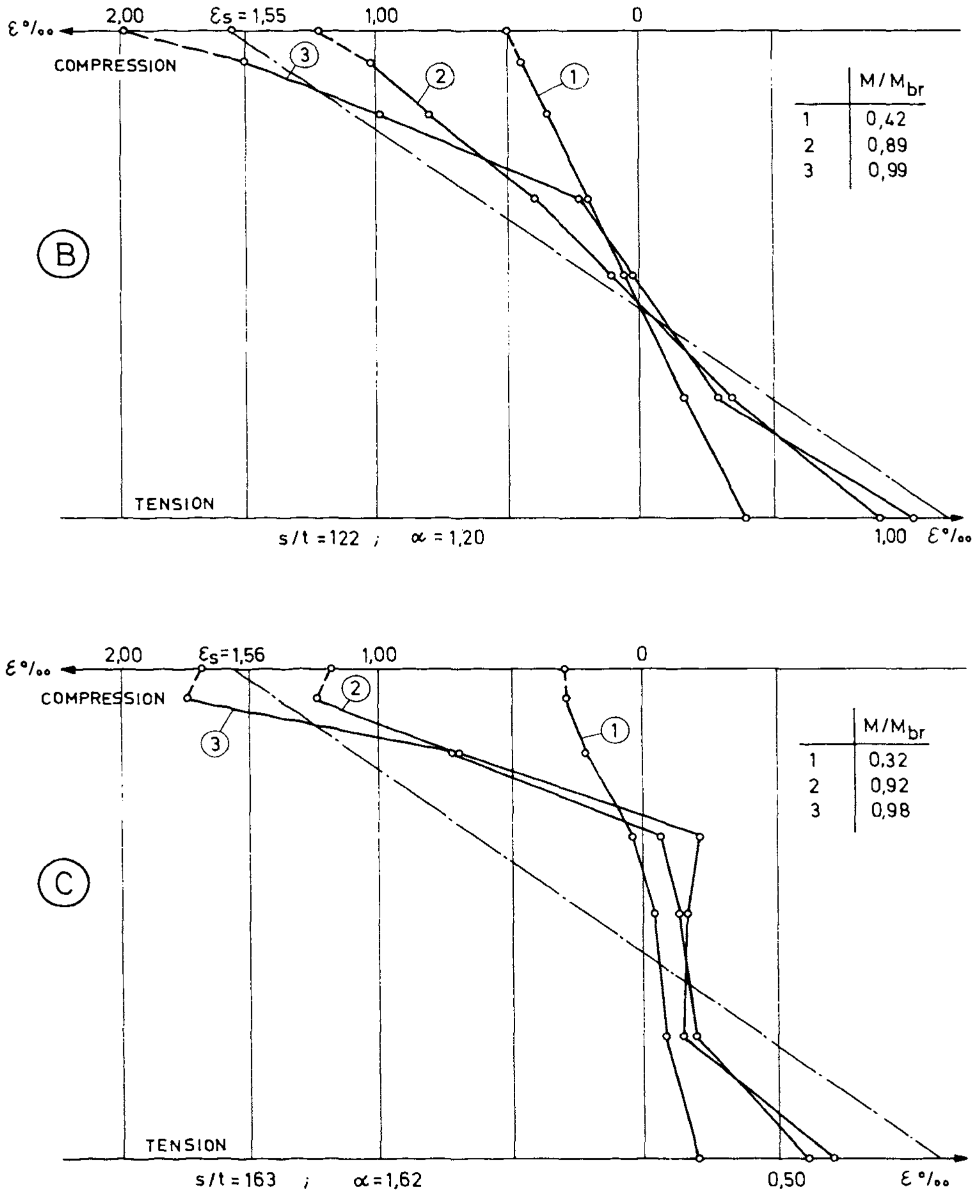


FIG. 4.4 Example of strain distribution measured over the cross section of the web (M_{br} = measured ultimate moment)

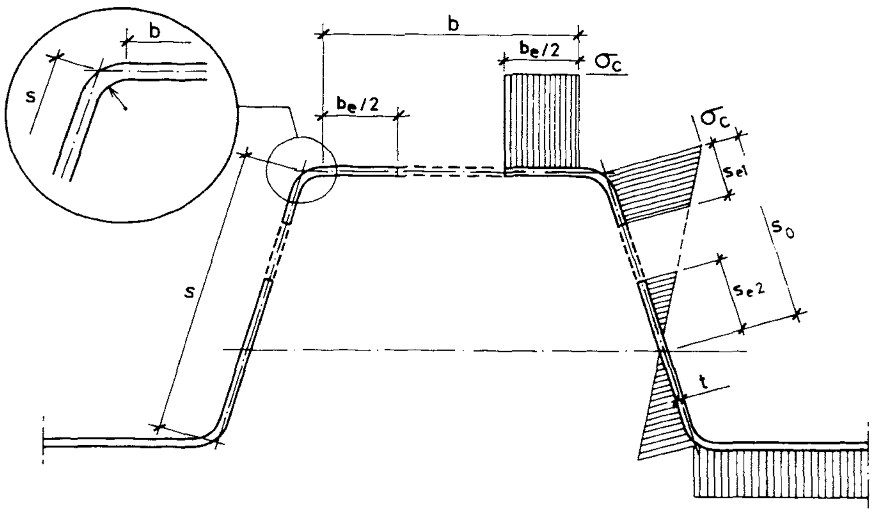


FIG. 4.5 Analytical model for determination of the effective width for a corrugated section acted upon by a bending moment

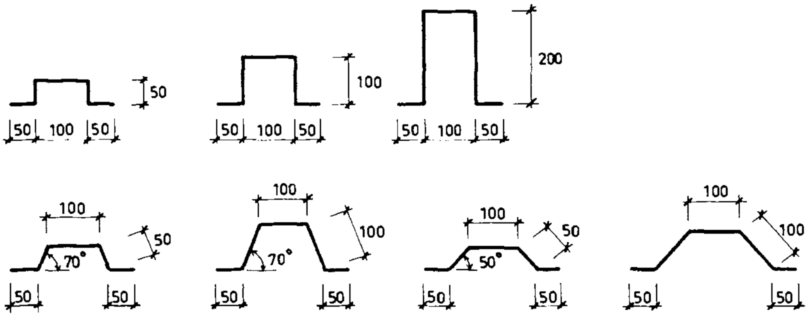


FIG. 4.6 Test specimens for support tests

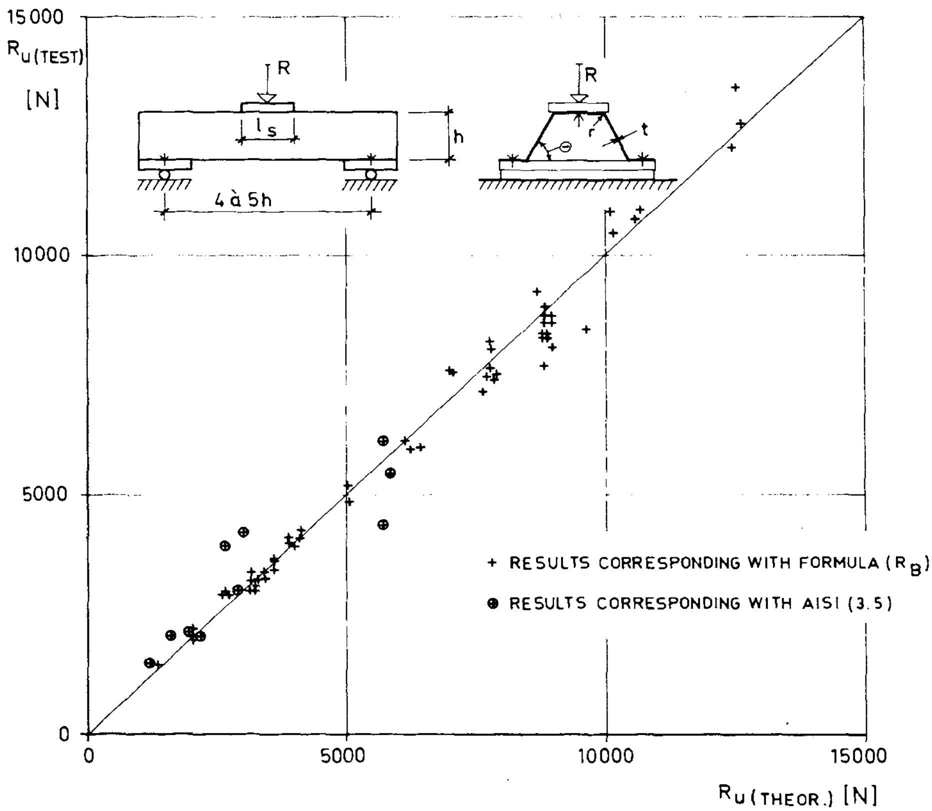


FIG. 4.7 Results from support tests (R_{test}) compared with calculated ultimate load (R_{teor})

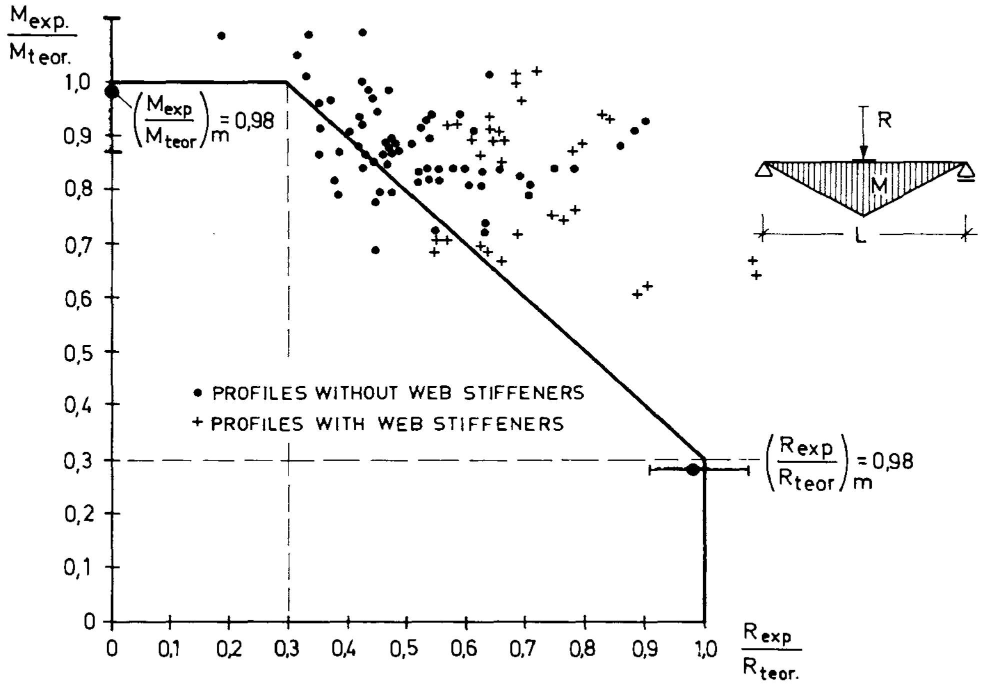


FIG. 4.8 Comparison of experimental and calculated ultimate values when the section is acted upon simultaneously by bending moment and concentrated load. The full curve shows the interaction relationship according to the Swedish code.

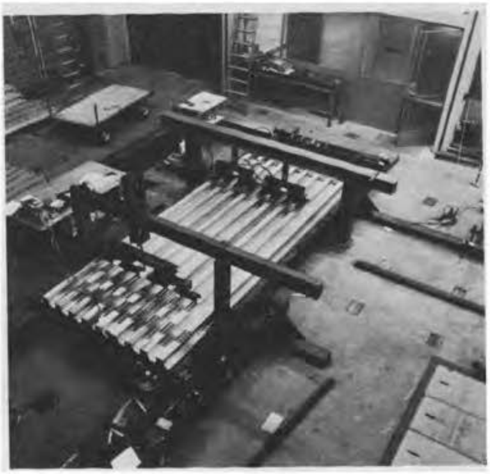


FIG. 4.9 Test frame for in-plane loading

FIG. 4.10 Global buckling of sheet panel comprising TRP 50

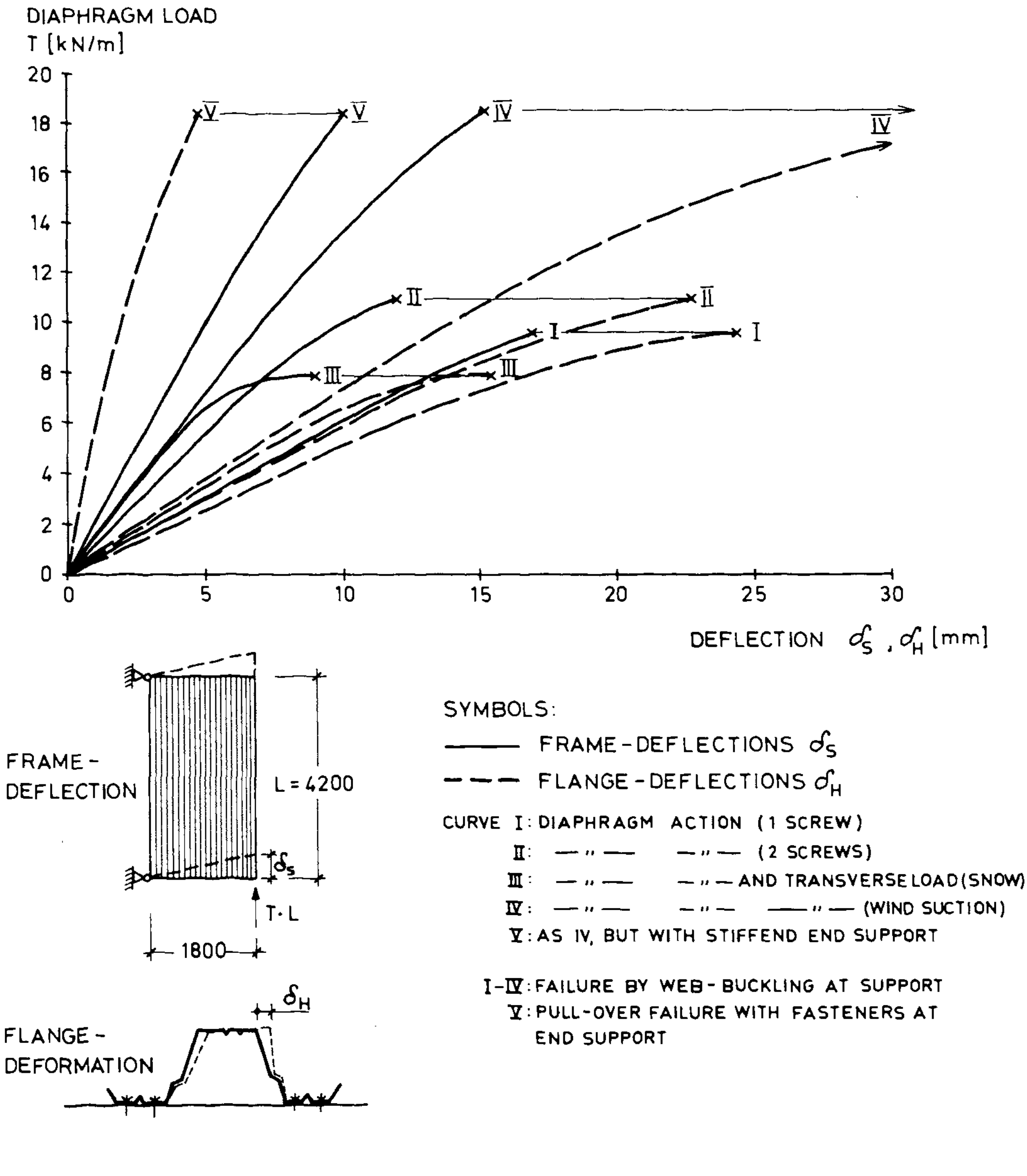


FIG. 4.11 Deformation curves for panel of TRP 110-0,65 for different load combinations

FIG. 4.12 Example of failure at end support due to web crippling

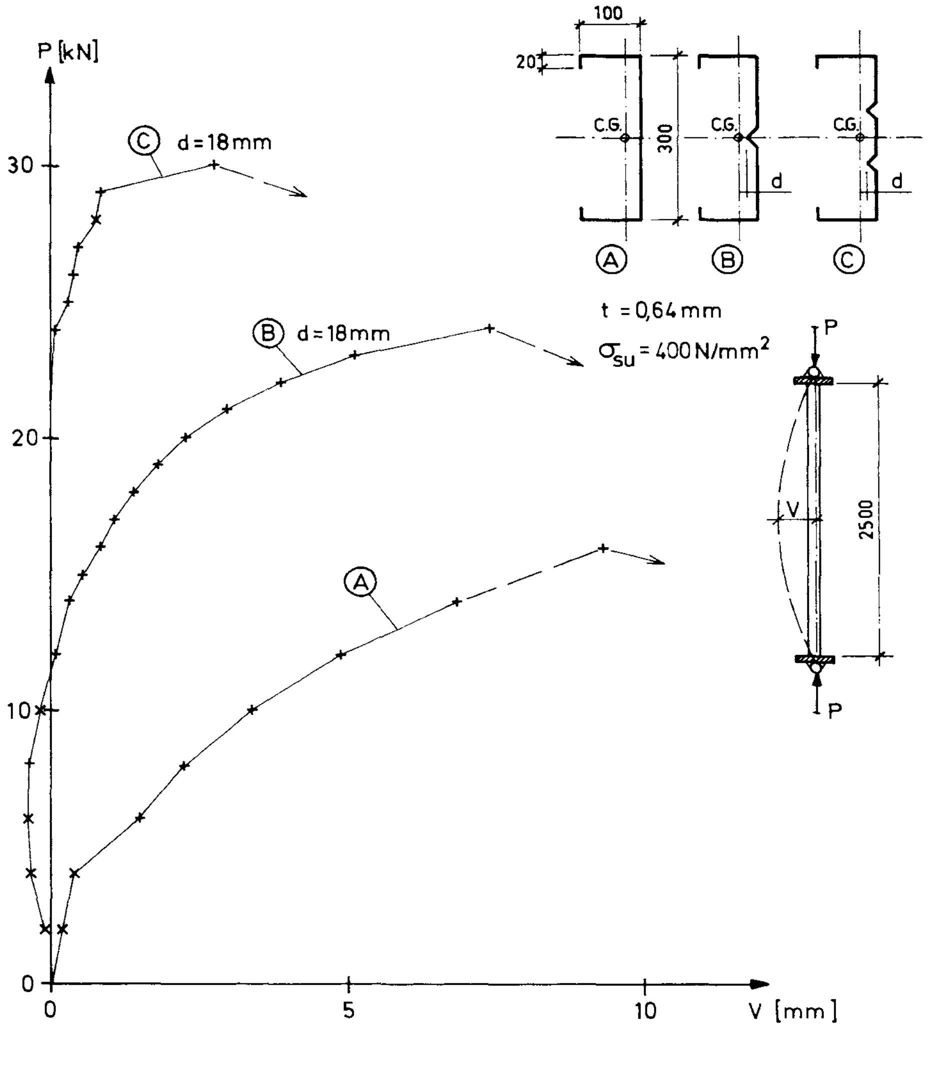


FIG. 4.13 Load-deflection curve for axially loaded channel sections. A without intermediate stiffener, B with one intermediate stiffener, C with two intermediate stiffeners

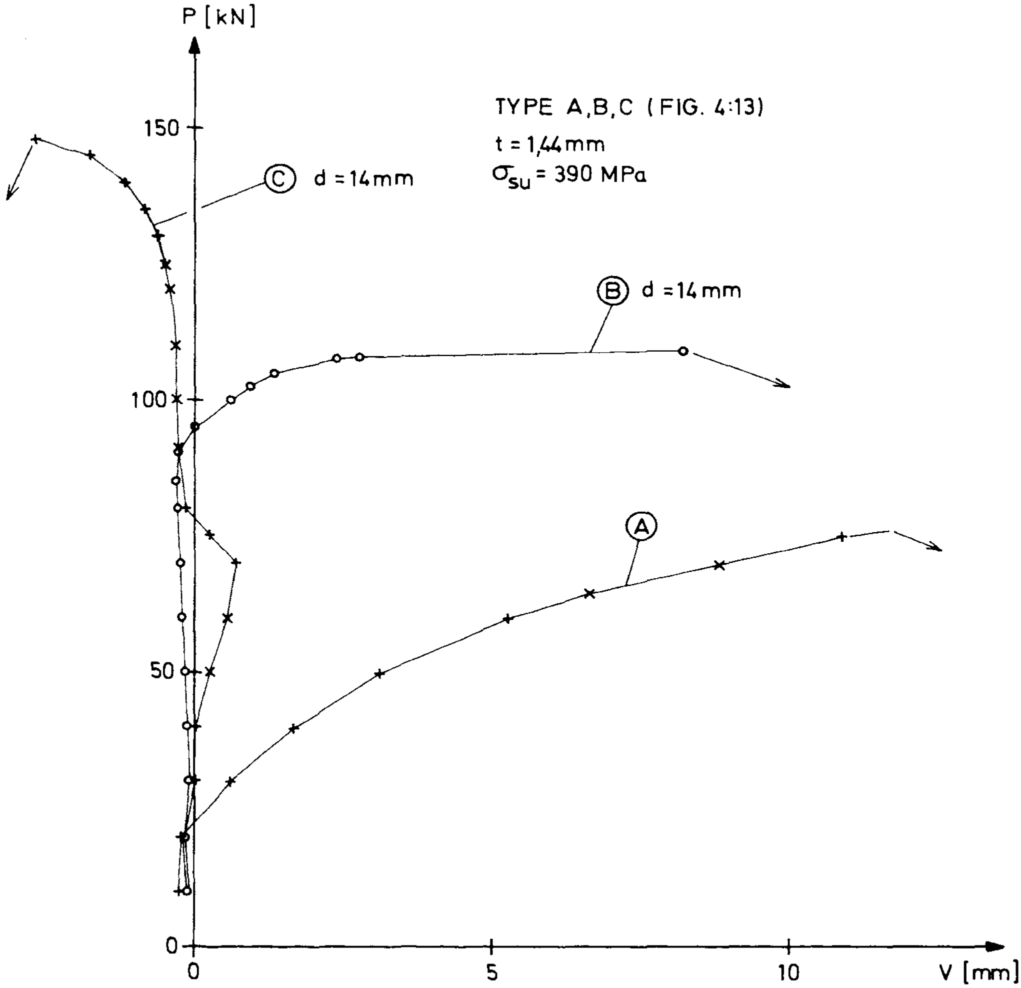


FIG. 4.14 Load-deflection curve for axially loaded channel sections

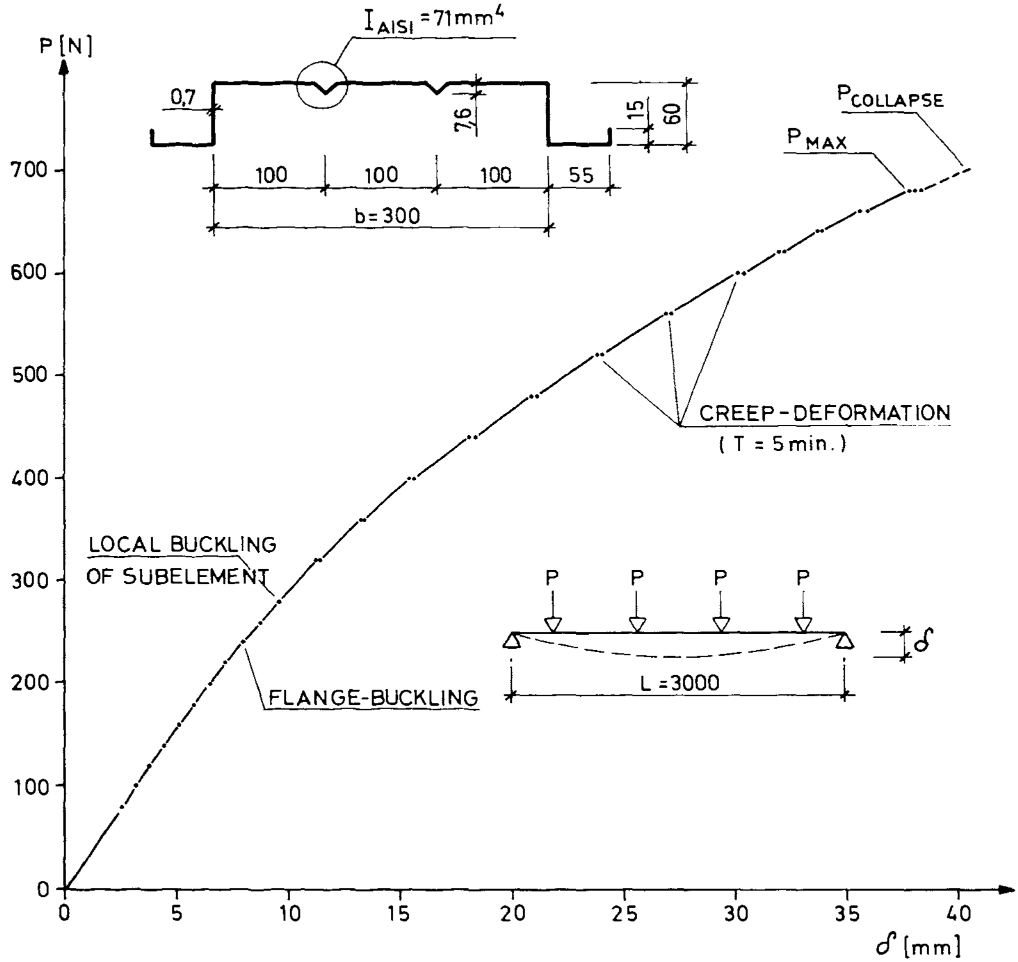


FIG. 4.15 Example of load-deformation curve for wall consisting of channel section panels acted upon by a transverse load

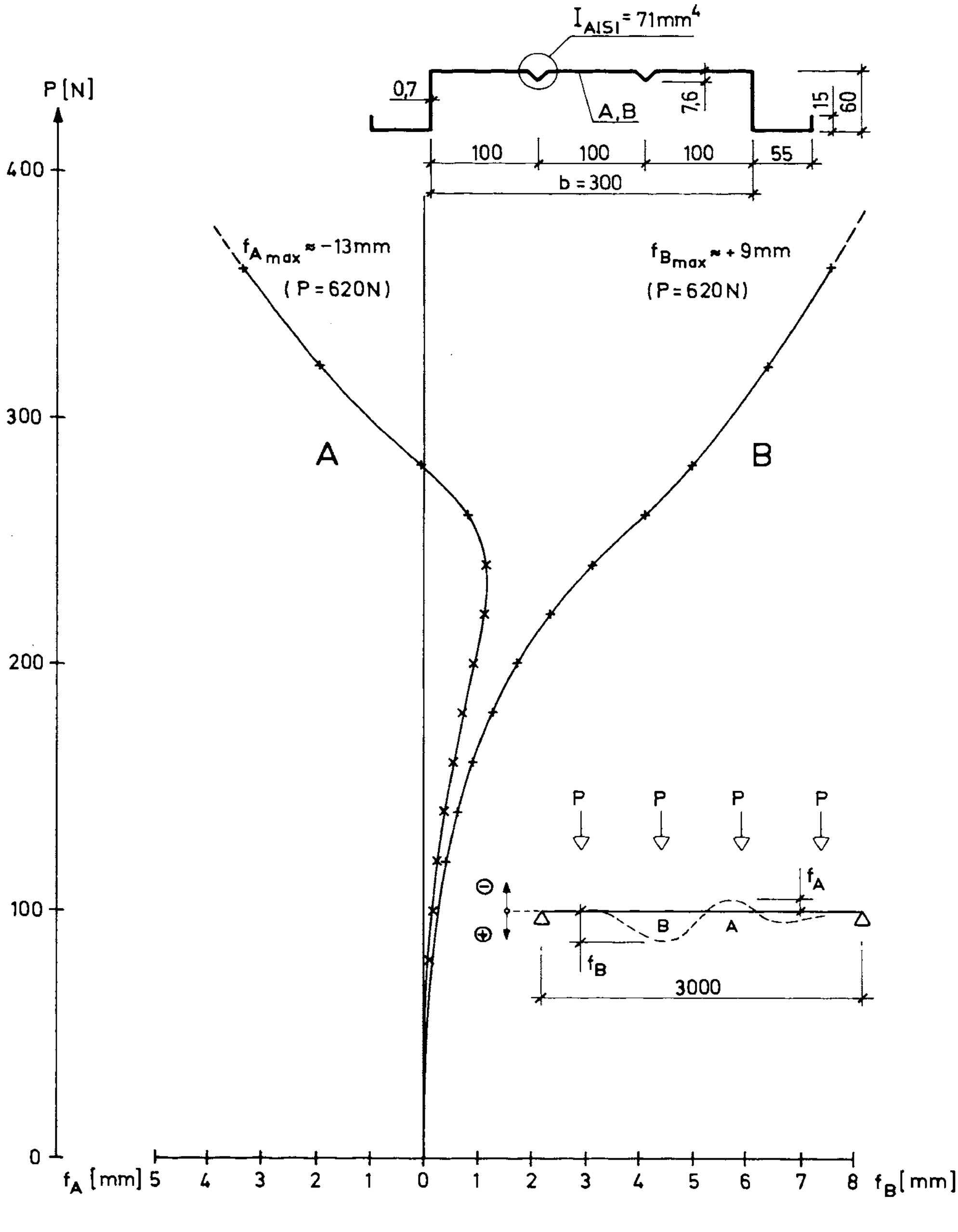


FIG. 4.16 Buckling configuration of flange with intermediate stiffener in channel section subjected to a transverseload

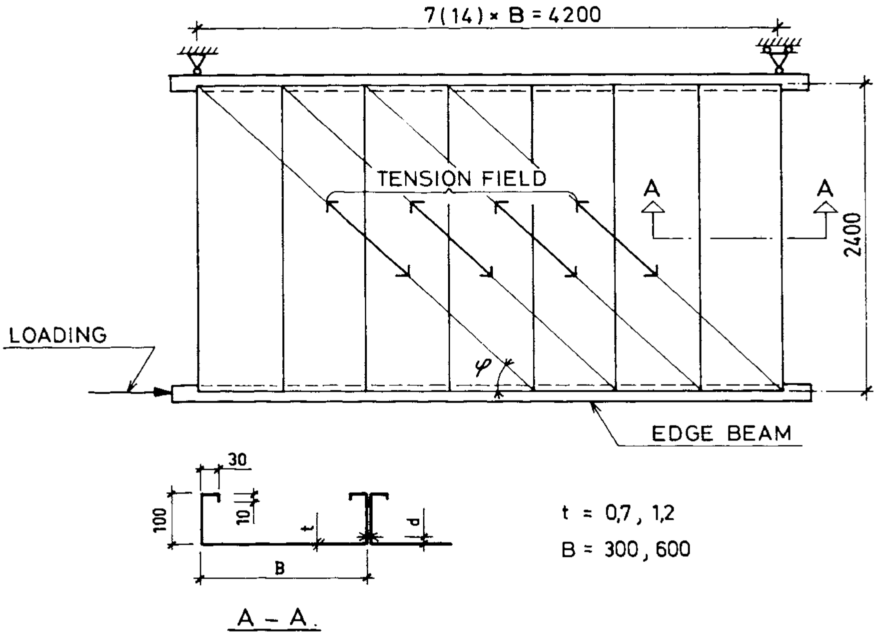


FIG. 4.17 Test rig for testing in-plane action

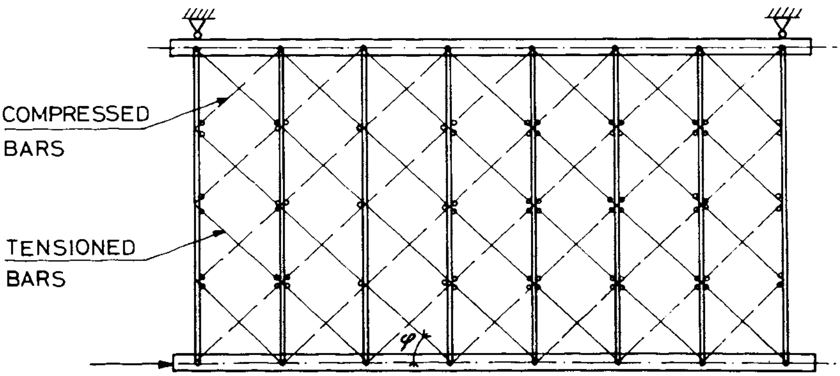


FIG. 4.18 Analytical model of the truss type for determination of the loadbearing capacity

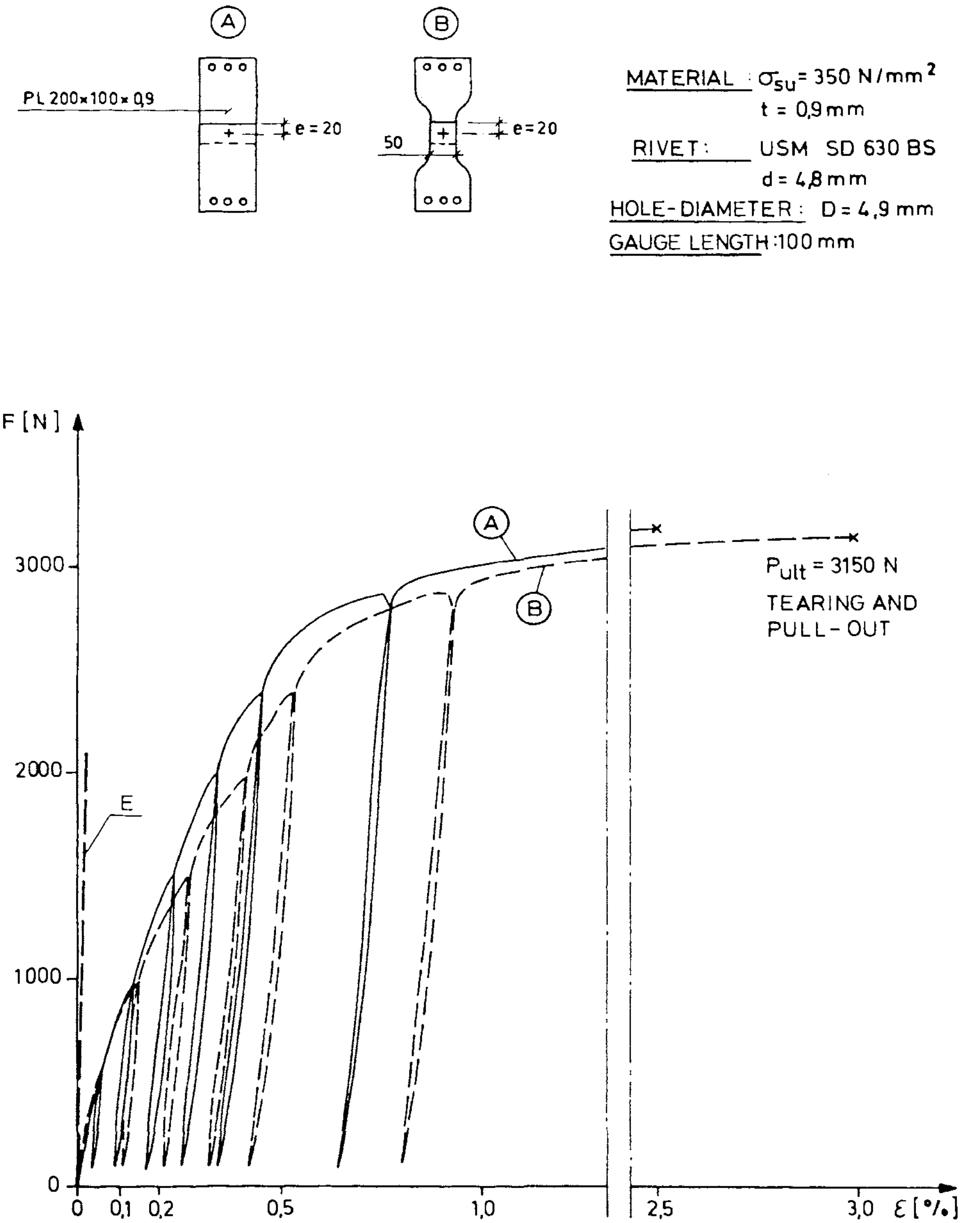


FIG. 4.19 Relationship between load level and deformation in riveted connection

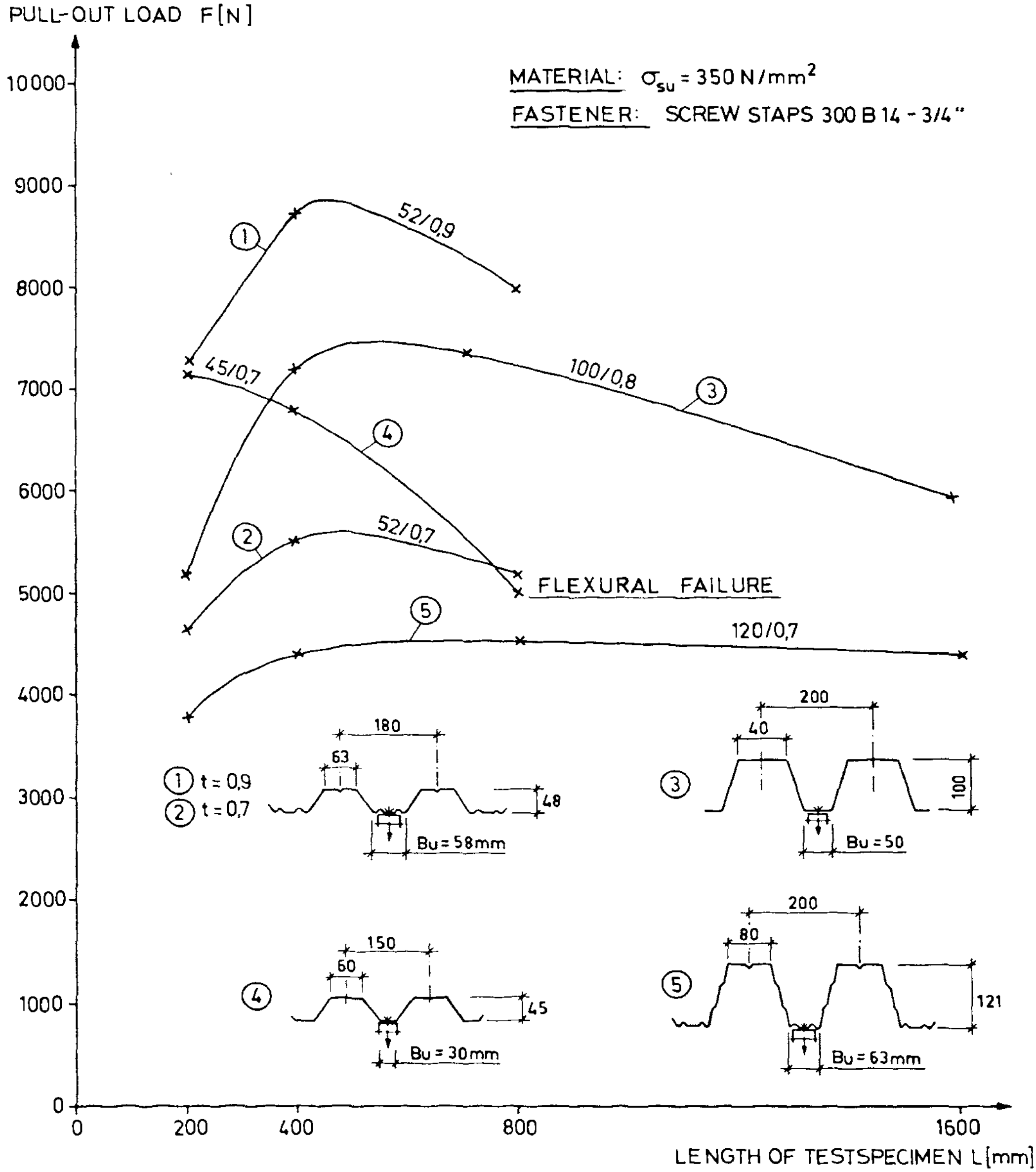


FIG. 4.20 Pull-out strength of corrugated sheeting

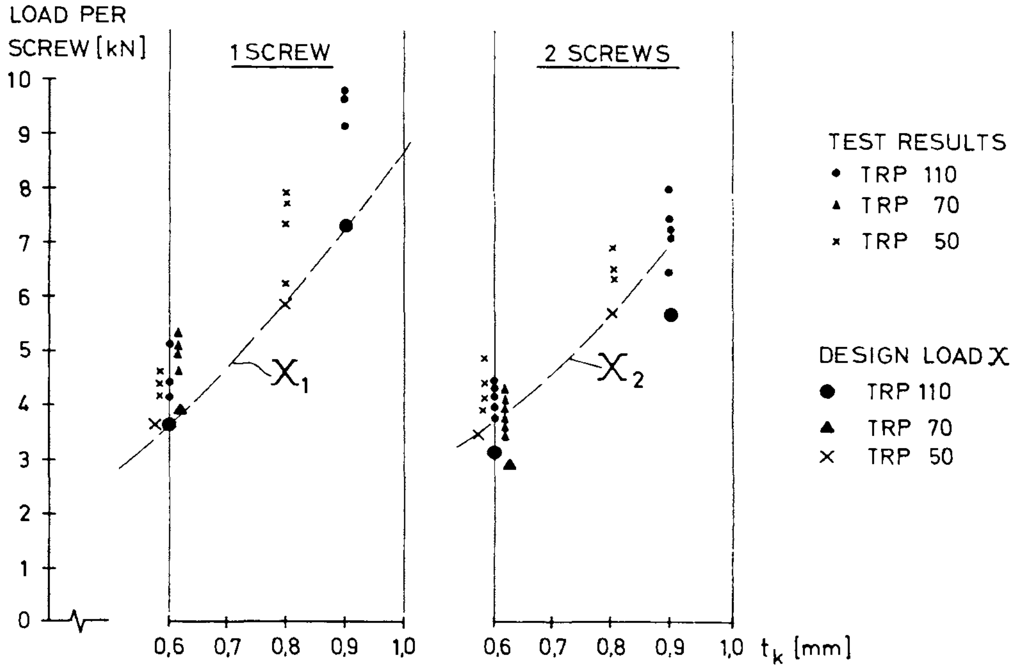


FIG. 4.21 Results of pull-out tests on TRP 50, 70 and 110 with 1 and 2 screws STAPS 300 B 14

TEST-SERIES	$R=\frac{P_{min}}{P_{max}}$	FREQU. Hz	TESTMATERIAL t_k [mm]	F_y [MPa]	QUAL.	FASTENER: RIVETS (USM) ϕ [mm]	MATERIAL; TYPE	
B	0,1	25	0,77	433	SUB350	4,8	STEEL ;	SD 630 BS
C	0,1	25	0,72	474	—"—	4,8	MONEL ;	LD 630 BS
D	0,1	25	0,74	467	—"—	4,8	ALUMINIUM ;	AD 66 BSLF
E	0,25	25	0,80	405	—"—	4,8	STEEL ;	SD 630 BS
F	0,50	25	0,77	463	—"—	4,8	STEEL ;	SD 630 BS

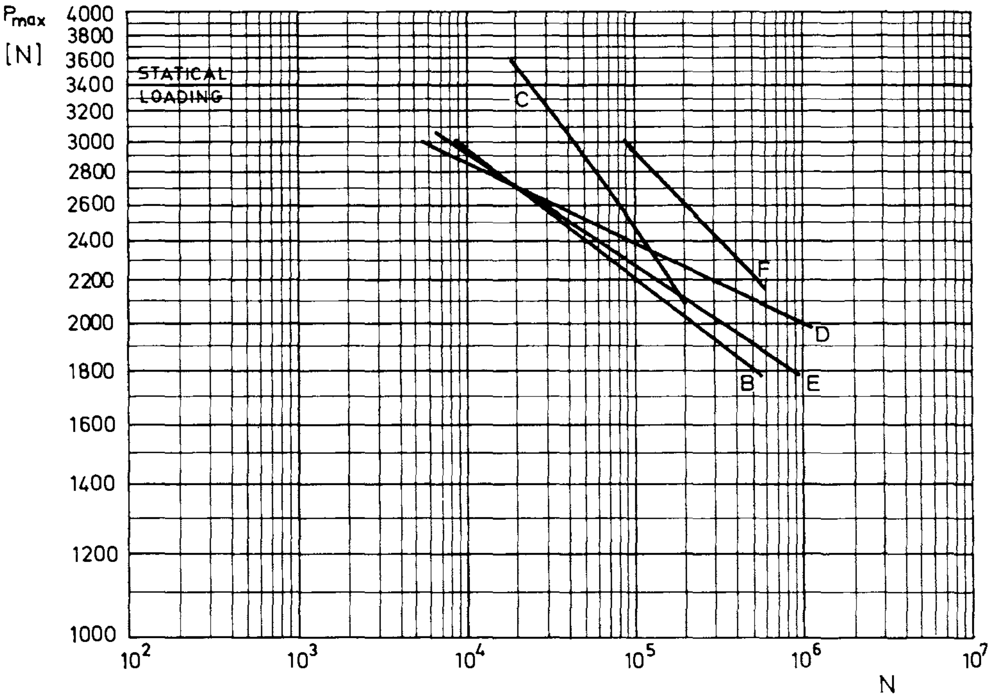


FIG. 4.22 Fatigue testing of sheet metal connections using different types of rivets

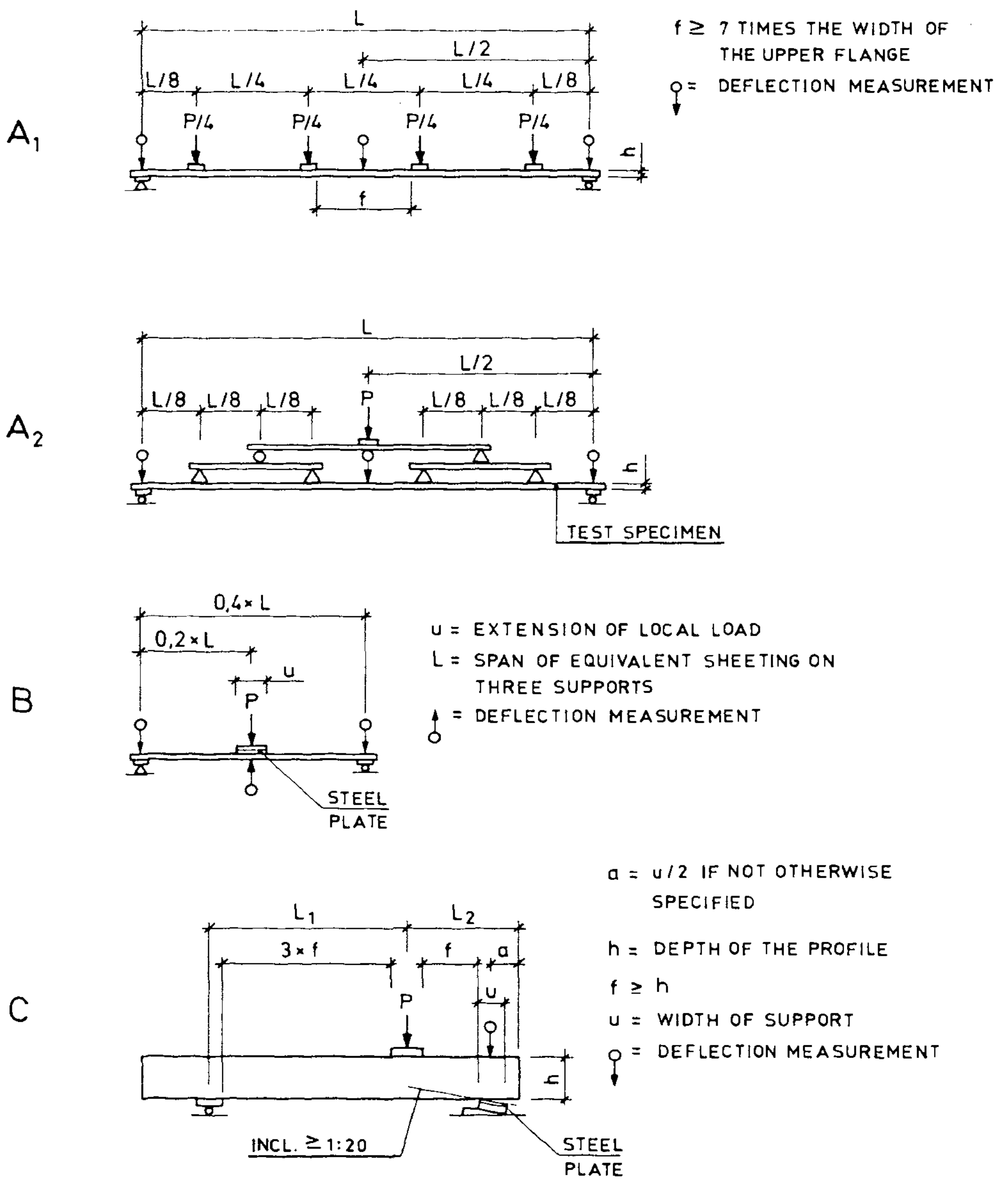


FIG. 5.1 Test rig for determination of span moment (A), support moment (B) and bearing pressure (C)

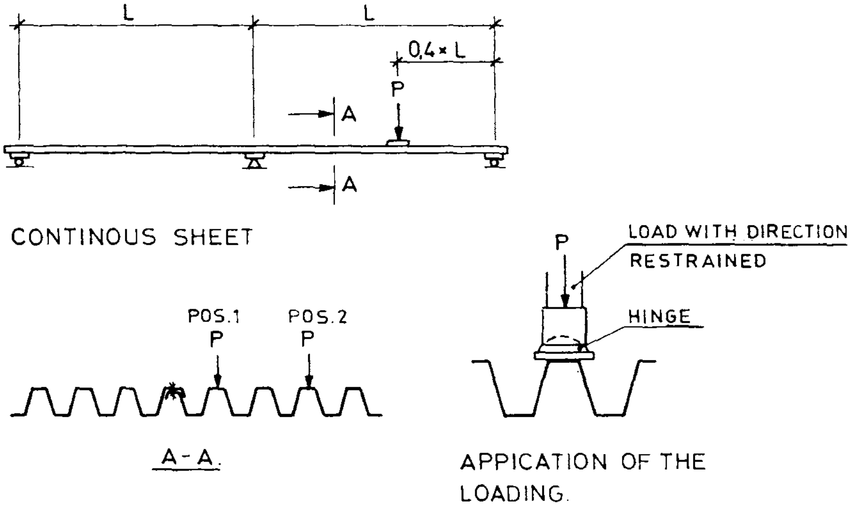


FIG. 5.2 Test rig for concentrated load

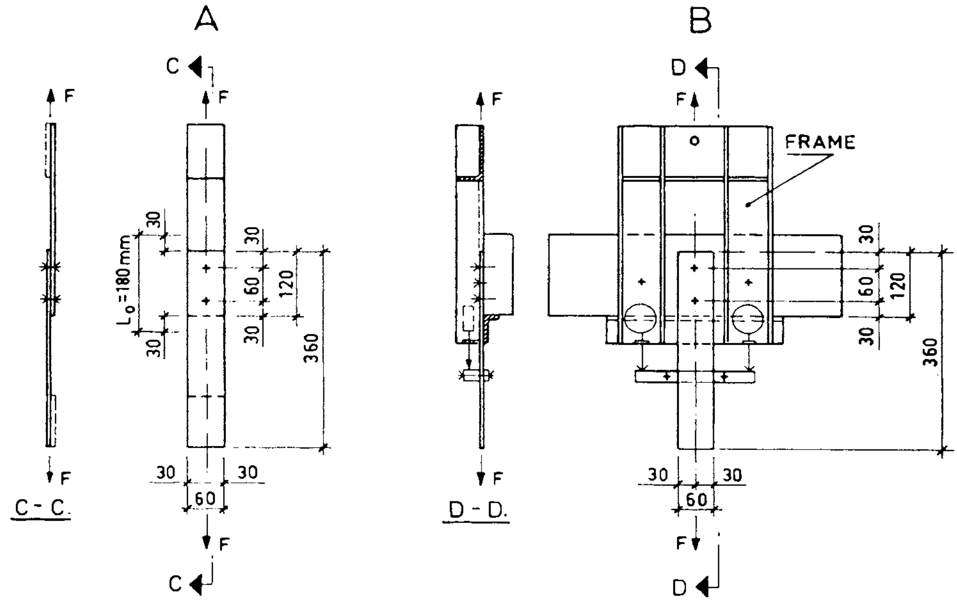
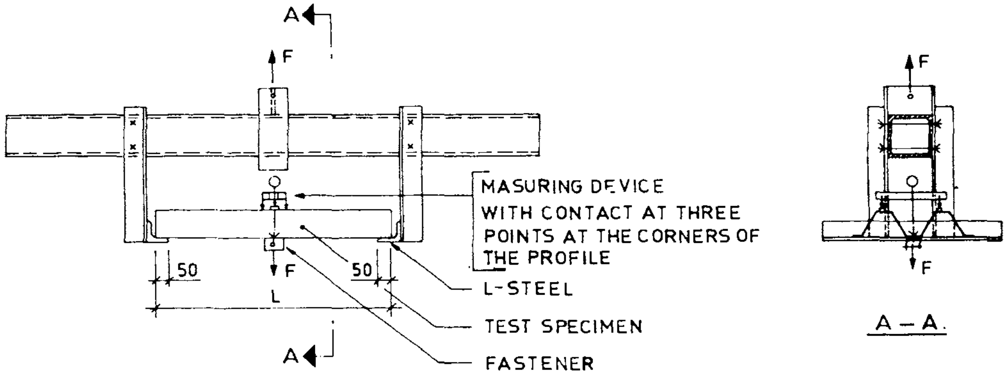


FIG. 5.3 Test procedure for connection in shear



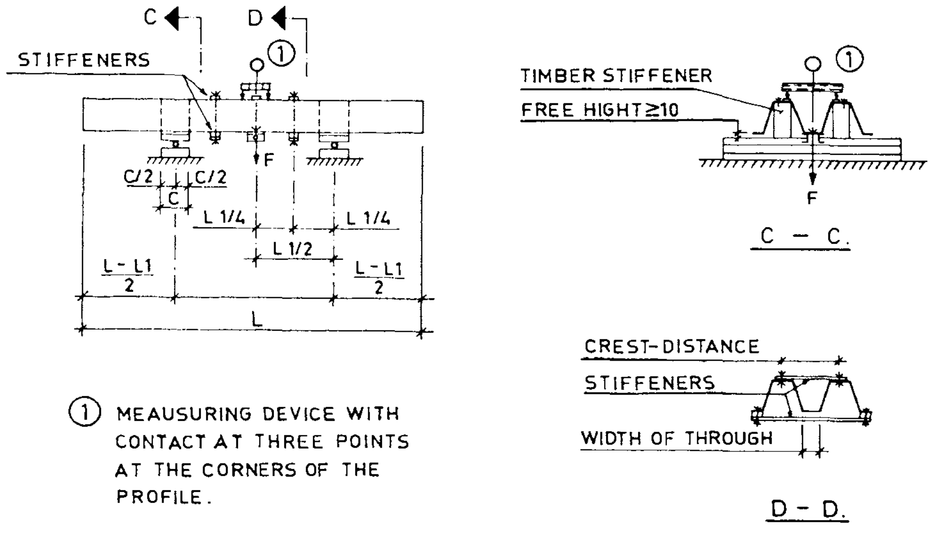


FIG. 5.4 Test rigs used in pull-out tests