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K. Basler

NATIONAL ENGINEERING CONFERENCE

1960 PROCEEDINGS

AMERICAN
INSTITUTE OF
STEEL
CONSTRUCTION

CONTENTS

THURSDAY MORNING SESSION

May 5, 1960

Presiding: DUANE T. MOLTHOP, President, Burkhardt Steel Co., Denver, Colo.

Recent Developments in the Use of High Strength Bolts

EDWARD R. ESTES, JR.

The most recent development in high strength bolts is the new "Specification for Structural Joints Using ASTM A325 Bolts." This new specification, which replaces the old specification in use since 1954, was approved by the Research Council on Riveted and Bolted Structural Joints on March 22nd of this year. Subsequently, it received official endorsement by the AISC on April 6th, and the Industrial Fasteners Institute on April 26th.

One of the most important provisions of this new specification is the recognition of the difference between friction-type connections and bearing-type connections when they are required to transmit load between connected parts as a shear type joint.

In shear connections where the engineer is concerned with the adverse effects of stress reversal, severe stress fluctuation, impact or vibration, or where slippage would be undesirable, he would specify that the joint be the friction-type. Many years of experience and research have shown that the high tension in the bolt permits transfer of load from one connected part to the other by friction between surfaces. For this type of connection, the Council has always required that the faying surfaces be unpainted. The allowable working stress for high strength bolts in this type connection remains the same as the value of a rivet in shear or 15 ksi on the nominal cross section area.

However, the Council also realized that movement of the connected parts bringing the bolts into bearing against the sides of their holes is not detrimental in many installations. In the new specification, as in the old one, faying surfaces in these connections may be painted since it is anticipated that the bolt will slip into bearing. Furthermore, when the threads are excluded from the contact surface shear planes, the shear strength of A325 bolts is greater than that of hot driven rivets. The Council approved a higher working stress value based on tests on large bolted joints which have been conducted at Lehigh University under Council sponsorship for the past several years.

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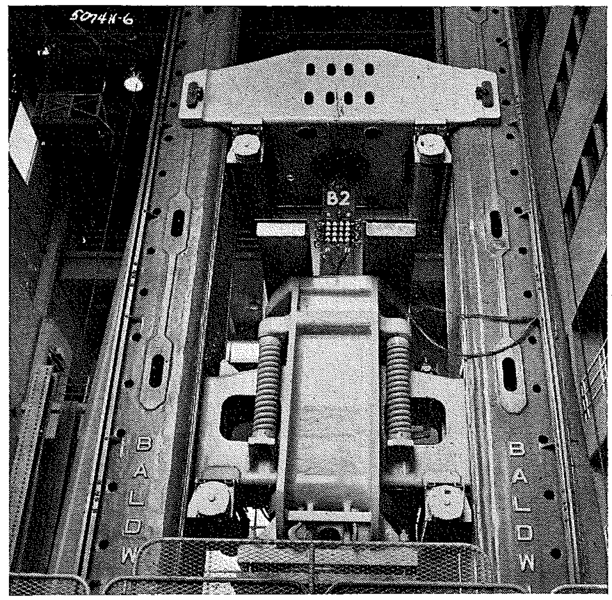


FIGURE 1

This study showed just how much stronger high strength bolts are in resisting actual shearing forces and what the effects of the higher stresses in the bolts are upon the strength of the connected parts.

The butt joint shown in Figure 1 was made up of four 1-inch plates and contained twenty-five 7/8-inch high strength bolts. The joints were tested in a 5 million pound universal testing machine. In determining the strength of the bolt which could be counted on to develop a balanced joint, as the number of fasteners in a joint was decreased, the type of failure changed from plate failure (Fig. 2) to bolt failure (Fig. 3).

The description of test joints (Fig. 4) shows that the tension-shear ratio varied from 1:0.75, the tension-shear ratio permitted in the present specification when a bolt is substituted for a rivet, up to a value of 1:1.15. With the exception of the riveted joint BR-2, all of the joints shown (Fig. 5) developed the coupon strength of plate material. The black bars indicate that although a bolt

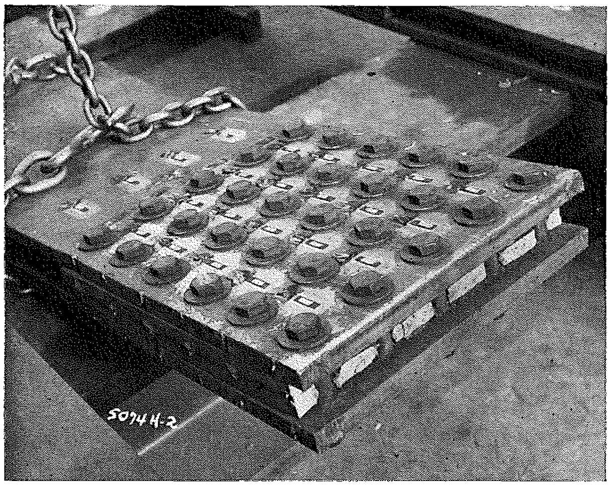


FIGURE 2

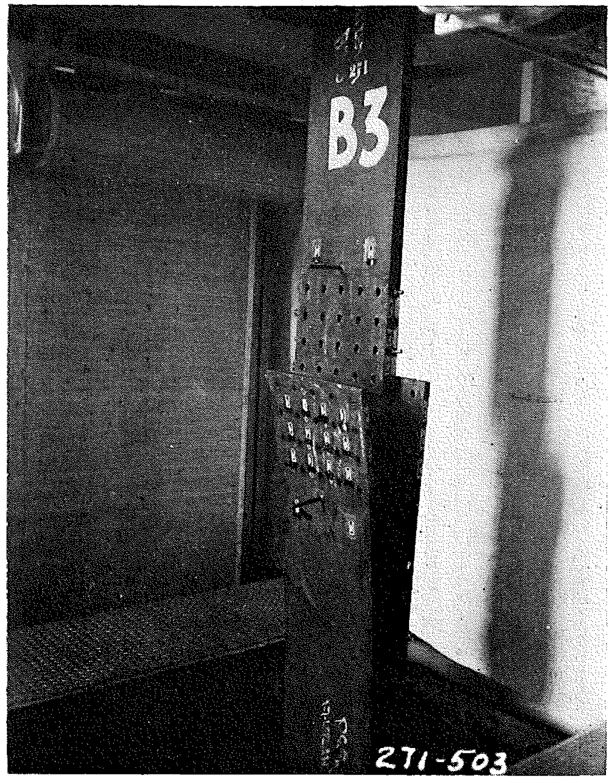
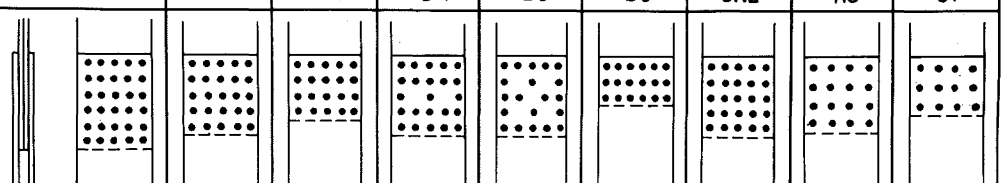


FIGURE 3

failure occurred the strength of the plate was developed and that with A325 bolts a tension-shear ratio of 1:1.1 gives a balanced design. Thus, when 20 ksi is permitted on the net section in tension, 22 ksi is permitted on the nominal cross sectional area of the bolt in shear.

However, in these large joints the full body of the bolt was present in all shearing planes. Additional tests have shown (Fig. 6) that if threads are in one shear plane of a double shear connection the strength of the bolt is reduced by 15% and if they are in both shear planes the strength of the bolt is 70% of the bolt in which no threads are in the shear plane. At a 100% load when the threads are not in a shear plane, the shear ratio as stated previously is 1:1.1. Thus for the case when threads are present in both shear planes the working value would be equal to 0.70 of 1.1 or approximately 75% of the tensile stress. In bearing-type connections then, when threads are in the shear planes the allowable working

DESCRIPTION OF TEST JOINTS



	B1	B2	B3	B4	B5	B6	BR2	A3	G1
PLATE MATERIAL	MAIN PLATE: 2 PLs 18"x1" ASTM—A7 LAP PLATES: 2 PLs 18"x1" ASTM—A7								
NUMBER OF 7/8" A325 BOLTS	30	25	20	23	20	18	25—7/8" A141 Rivets	16—1"	12—1 1/8"
T:S RATIO	1:0.74	1:0.89	1:1.11	1:0.96	1:1.11	1:1.15	1:0.89	1:1.10	1:1.11
GAGE, g	3 5/8"	3 5/8"	3 5/8"	3 5/8"	3 5/8"	3"	3 5/8"	4 1/2"	4 1/2"
g/d	3.87	3.87	3.87	3.87	3.87	3.20	3.87	4.24	3.79
PITCH, p	3 1/2"	3 1/2"	3 1/2"	3 1/2"	3 1/2"	3 1/2"	3 1/2"	4"	4"
p/d	3.73	3.73	3.73	3.73	3.73	3.73	3.73	3.76	3.37

"d" is the diameter of the drilled hole

FIGURE 4

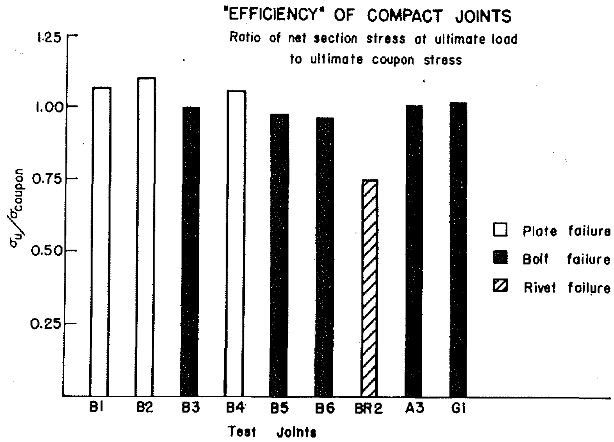


FIGURE 5

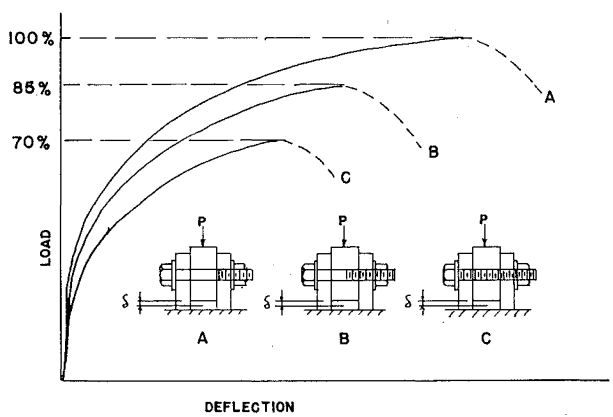


FIGURE 6

stress for bolts will be 15 ksi, although an intermediate value could be used for the case of threads in one plane only.

Tests conducted at the University of Illinois and reported in the ASCE transactions have shown that bearing pressure for rivets in double or single shear has no effect on the strength of the connected parts, so long as this pressure does not exceed 2¼ times the tensile stress on the net area. No difference in allowable bearing value seems to be justified when high strength bolts are substituted for rivets, so the allowable bearing stress in bearing-type joints, whether single or double shear, is 45 ksi. Bearing stress need not be investigated in friction-type joints.

In the previous specification no working stress was recommended for tension. Since then, tests have shown that at a working value of approximately ⅔ of the initial tightening force the high strength bolt will experience little if any change in stress and that the fatigue strength under this condition is not adversely affected. Maintaining an adequate factor of safety and relating to nominal rather than net or stress area, the recommended working stress for A325 bolts in tension is 40 ksi.

This summary (Fig. 7) shows the allowable working

ALLOWABLE STRESSES

A 325 BOLTS

based on nominal area

	AISC	AASHO AREA
	ksi	ksi
TENSION	40	36
SHEAR		
Friction-type	15	13.5
Bearing-type	22	19.8
BEARING		
Bearing-type	45	40.5

FIGURE 7

stresses permitted in the new specification related to the AISC basic design stress of 20 ksi and the AREA and AASHO basic design stress of 18 ksi. All these stresses apply to the nominal area of the bolt.

Another very important revision in this new specification is the permission to use any one of three styles of bolts, and two types of nuts (Fig. 8). It has long been realized that it would be desirable to have the same size bolt head and nut enabling the iron worker to use only one size wrench or socket for a given bolt assembly. The use of a finished, heat-treated, hexagon nut with the same dimension across flats as the regular semi-finished hexagon head bolt is now permitted. Furthermore, it has been realized that the elimination of one or both washers would result in considerable economy in a bolted connection. Hence, the bolt manufacturers are now producing an A325 bolt with a heavy semi-finished hexagon head; one which has the same dimensions across the flats as the heavy semi-finished hexagon nut, but with the same height of head as the regular semi-finished hexagon bolt.

At the Engineering Conference in St. Louis two years ago the interference-body bolt was introduced. The use of this bolt which reduces slip in the joint has increased considerably during this period of time. Since the interference-body prevents the bolt from turning in tightening only one man is needed to install this connector. The head has the shape of a button head rivet with the exception that it is flattened to the same height as the regular hexagon head bolt.

As was the case in the previous specification all the bolts are identified by three radial lines and the manu-

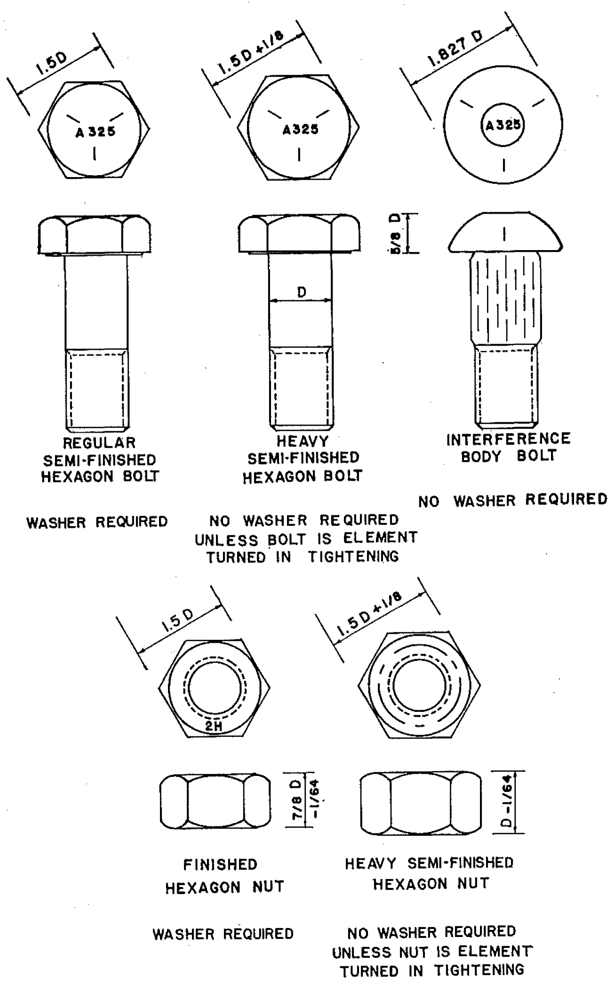


FIGURE 8

facturer's mark. Heavy semi-finished nuts are identified by three circumferential marks or in some instances by the number 2 and the manufacturer's mark. The finished hexagon nuts are identified by the symbol 2H.

The Council felt that the minimum outside washer diameter required could be reduced since very little pressure is exerted along this outer edge. The outside diameters now called for are the smallest available without change of thickness.

Another important development is a start toward the elimination of washers in high strength bolted joints. Tests have already shown that the elimination of both washers do not affect the ability of the bolt to maintain its clamping force. However, in order to maintain a suitable torque tension ratio when a calibrated wrench is used, a washer is required under either the nut or bolt head, whichever is turned in tightening. The Council also decided to leave the washer under the regular hexagon head bolts and finished nuts for the fear of oversized holes reducing the bearing area under these smaller bolt heads and nuts. A test program is under way to show that the slight amount of galling which might occur when the element turned in tightening rubs against the connected plate, even with over-sized holes, has no adverse effect upon the fatigue strength of the joint. In the interim, washers are not required under heavy hexagon head bolts or nuts if they are not turned in tightening, and are never required under the head of the interference-body bolt.

The new specifications require that the A325 bolt be installed to at least the proof load of the bolt in tension. Experience and research have proved that the greater the tension in the bolt the more satisfactory the behavior of the connection. No approximate equivalent torque values are given as in the past. Instead, it requires that the torque or impact wrench be calibrated in a device capable of measuring the actual tension produced by a given wrench effort.

Calibration studies show that the turn-of-the-nut method as prescribed in the specification consistently produces tensions above the minimum proof load. During assembly of some of the large bolted joints a record was kept of bolt elongation (Fig. 9). In each connection the plates were pulled up tight with A325 fitting up bolts (solid squares). Bolts in the remaining holes (open squares) were then given a half turn from the snug position. Snug position is defined as the point where the impact wrench begins to impact after the nut has been run up on the bolt. The tension versus bolt elongation curves are plotted for the case where tension was produced in the bolt by rotating the nut and the case where the bolt assembly was pulled in direct tension in a testing machine.

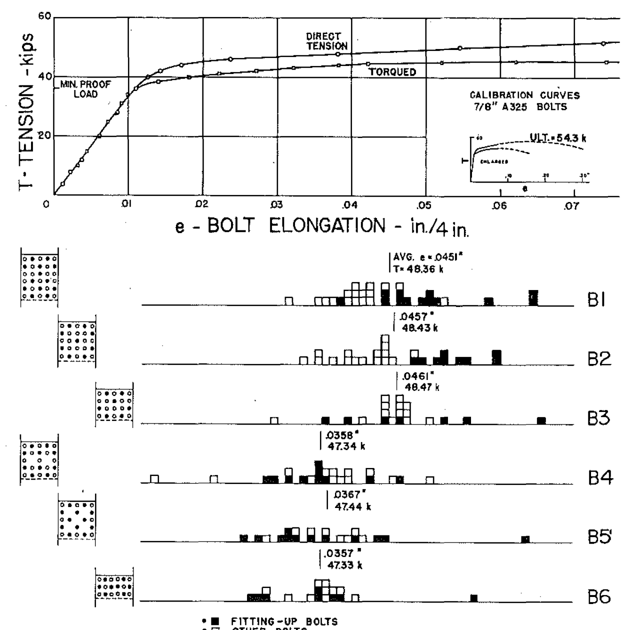


FIGURE 9
Bolt Tension Distribution

In these installations the elongation of the bolt was measured while tightening and recorded in its relative position parallel to the elongation axis. By projecting upward from a given bolt location the corresponding bolt tension is found from the calibration curve. The most consistent bolt tension is obtained from an elongation control such as provided by the turn-of-the-nut method. This is due to the nearly horizontal portion of the curve when the proof load has been exceeded. Even though the bolt is in the inelastic range an adequate margin against breaking is present.

With the requirement that threads be kept out of the shearing plane in order that the benefit of the full strength of the A325 bolt be realized in bearing-type joints, it became apparent that it would be necessary to reduce the thread length of the A325 bolt and to require closer manufacturing tolerances. Under this new specification, the manufacturers will furnish bolts with a slightly shorter thread length than heretofore. Full threads will be excluded from all shear planes except where there is a thin outside part adjacent to the nut. Some thread runout in the shear plane is permissible without requiring a reduction from the permitted 22 ksi stress.

A Commentary accompanies the new Specifications to provide guidance in their application.

With the continuing increase in the use of high strength bolts it appeared that it might be possible to justify their use in the shop as well as in the field. Consequently, time and motion studies have been sponsored by a bolt manufacturer in cooperation with one of the larger fabricators. A sufficient number of identical joints were available for comparing bolting and riveting operations. The procedures followed in fitting up and riveting, with the necessary intermediate steps of moving and storage, are familiar. In case of shop bolting the use of fitting up bolts and tack welding is eliminated. The fitting up and bolting all take place at one position, and the material is then moved directly to the shipping yard resulting in a release of storage space for other work. One particular study of trusses, bracing members, and beams (Fig. 10) showed a saving of almost 40% in operating costs with shop bolts instead of rivets. However, the cost of the bolts, nuts and *two* washers was 175% greater than the cost of rivets. However, this represented a resultant savings of 2% when bolts were used. With one washer eliminated as will now be permitted, the saving would have amounted to 10% in the case of bolting. If both washers are eliminated, as we have every reason to expect within the very near future, the savings by bolting would have been 17%.

TOTAL COST: *Bolting vs Riveting*

TRUSS | BRACE | BEAM

	Rivet	Bolt	Difference $	Difference %
OPERATING COST • Riveting: Rivet fit-up, handling riveting, shop overhead	$870.48			
• Bolting: Bolting, handling, shop overhead		$524.11	–$346.37	–39.8
MATERIAL COST • Riveting: Rivets, rivet heating, fit-up bolts, welding rod	185.43			
• Bolting: Bolts, nuts, washers		510.64	+325.21	+175.4
TOTAL: Labor, Overhead, Material Riveting: Bolting:	1055.91	1034.75	–21.16	–2.0
DEDUCT: *Washer under head* Riveting: Bolting:	1055.91	–79.55		
		955.20	–100.71	–9.5
DEDUCT: *Both Washers* Riveting: Bolting:	1055.91	–79.55		
		875.65	–180.26	–17.1

FIGURE 10
Summary of total costs recorded in shop studies.

This study was based on a design using the old specification in which one bolt was substituted for one rivet. Under the new specification ⅓ of these bolts could have been eliminated. Hence, there will be an even greater saving in the use of A325 bolts in the *shop* under the new specification.

The Research Council on Rivet and Bolted Structural Joints is continually investigating the high strength bolted joint; and when improvements are found to be satisfactory they are incorporated into the specification. While a new development may appear to be uneconomic at first, it should not be excluded for that reason alone. It may be that its permitted use may eventually prove to be economical and hence justified.

The effect of cooperation between fabricators, bolt manufacturers, consulting engineers and researchers, in studying the A325 bolt and in preparing this new specification will be realized in the near future, in the form of more economical structural steel designs.

Truss Bridge Project at Northwestern University

JOHN F. ELY

What It Is

The Truss Bridge Research Project consists basically of a 100-ft., half-scale steel truss bridge (Fig. 1) resting upon a massive reinforced-concrete test bed, with an arrangement for placing loads of varying magnitudes and patterns upon the structure, together with the instrumentation necessary to measure and record the effects.

The Project had its origin as long ago as 1920, when Professor Parcel, then at the University of Minnesota, is said to have suggested that a good way to find out how various members of a structure act under destructive loads might be to build a bridge in which selected members could be destroyed under the end conditions which are encountered in service.

This would overcome the obvious disadvantages of testing existing highway and railway structures, which must not be damaged during the tests, or of testing built-up structural members in laboratory testing machines wherein the unknown end conditions cannot be realistically duplicated.

Although many people agreed that this was a good idea, nothing came of it until 1952, when Committee 15 of the AREA decided to do something about it. During the next four years, the bridge was designed by Prof. L. T. Wyly, the foundations were begun in April of 1957, the bridge was erected in November of that year, and a building to house the Project was completed in August, 1958. In the Spring of 1959 an hydraulic system for loading the bridge was installed, and the first loads were imposed on the structure in June. Since that time a series of operational and calibration tests have been run, a research program established, and initial investigations of end-post failure have taken place.

The test bridge is a half-linear-scale model of a 200-ft., 8-panel, Warren type, double-tracked railway bridge. It is so arranged that the floor system may be changed, the portal bracing altered, and a highway-bridge configuration achieved when desired. A departure from normal bridge construction has been made, wherein the two trusses were not identical—the east truss, called Truss B, contains members with the angles turned inward, and with perforated cover plates, while the west truss, called Truss A, is built in the traditional manner, with angles turned out and the members laced.

Each of the trusses is built of Medium Manganese steel, except for the member being tested, which is composed of ordinary A7 carbon steel. The difference in yield points of these two materials permits the test members to be destroyed without serious damage to the remainder of the structure.

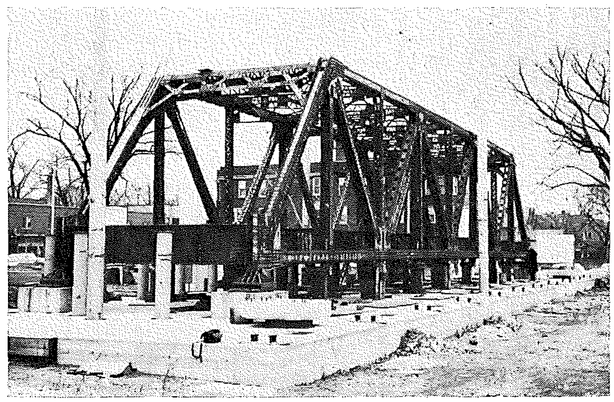


FIGURE 1

All joints being fastened with high-strength bolts, it is possible then to remove the destroyed member and replace it with another of different design. A succession of members, of different design, can thus be tested in this particular position. When the desired information about this portion of the structure has been obtained, the test member will be replaced by a Medium Manganese member, and similar series of tests will be conducted on other members. It is contemplated that this program of research will require several years, and it is expected to lead to improved or extended column theory.

The Project is conducted as an activity of the Civil Engineering Department of the Northwestern Technological Institute. It is sponsored financially by the Association of American Railroads, the American Institute of Steel Construction, the U. S. Bureau of Public Roads, the Corps of Engineers of the U. S. Army, and by the University. A large number of commercial, engineering and industrial organizations and individuals have made substantial contributions of materials and services.

Total investment in the Project to date is something more than a half-million dollars—a measure of the faith and confidence of the sponsors and contributors in the merits of the Project and the results which may be expected from it.

How it Works

The principal operations are the loading of the bridge, the observation and recording of data, the processing of data, theoretical studies arising therefrom, publication of results, and the planning of further research activities.

The observation and recording of data will be accomplished by means of systems of electrical, mechanical and optical instrumentation, and by recording apparatus

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which forms a part of the hydraulic loading system.

The loading of the bridge will be done hydraulically.

Loading the Bridge

Loads are applied to the structure by fourteen 150-ton hydraulic jacks, located in pairs, bearing upon the stringers of the bridge at their connections with the floor beams (Fig. 2). They are energized by centrally-located pumps, regulated from one central control panel. The loads are closely controlled and accurately measured by automatic equipment.

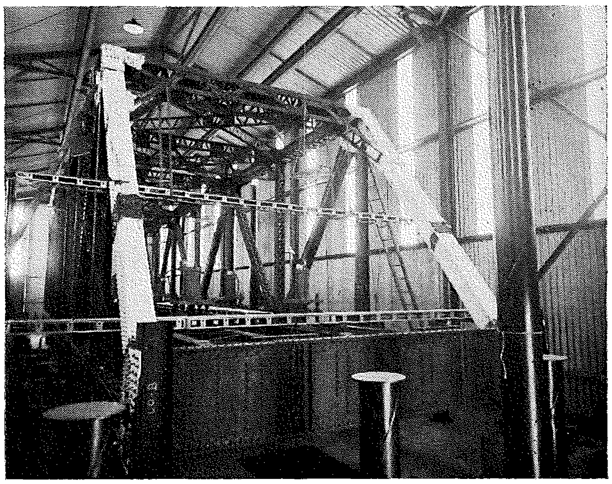


FIGURE 2

It was also necessary to provide for various loading patterns—to simulate, for instance, a heavily loaded train on one track and a lightly-loaded one on the other; or a locomotive at one end of the bridge, followed by a string of empty flat cars. This was accomplished by running two complete hydraulic circuits around the entire structure, terminating in headers at each jacking point. Individual jacks are connected to the appropriate header of either of the circuits with high-pressure hoses which are fitted with quick-disconnect couplings. Thus the jacks can be assorted between the two circuits, all connected to one, or not connected at all. It being possible to maintain either circuit at any pressure between 0 and 6000 psi, it will be seen that a very wide range of loading patterns is possible.

Controlling the Load

Inasmuch as it was desired, at the outset, to maintain the load to an accuracy of 1 per cent, it was early realized that hand regulation would not do the job. Therefore, an over-riding system of automatic controls was superimposed upon each hydraulic circuit.

The control system consists basically of two pneumatically-operated control valves in each of the high-pressure jacking circuits. One of these admits fluid into the circuit, the other relieves pressure. They assume various positions, between fully open or completely closed, in accordance with a pneumatic signal they receive from a Moore automatic controller.

The controller can be set to maintain a desired pressure in the jacking circuit. It receives information from a sensing device in the jacking circuit as to the pressure being supplied to the jacks. It compares this information with the desired pressure, and causes the control valves to open or close so as to correct any difference. A variation of as little as 0.1 per cent of system pressure will cause this instrument to function.

This ingenious and highly-precise instrument, along with other operating controls and load recording apparatus is mounted on a central control panel located alongside the bridge structure, from which all loading operations are carried on.

The net result is that loads upon the bridge can be imposed, maintained, and controlled to a precision of 0.1 per cent—ten times better than was originally hoped for, and better than most laboratory machines—and it was done entirely with commercial components, bought "off the shelf" and integrated into a smoothly functioning loading system. The hydraulic system was designed by Mr. Gerald C. Ward, Administrative Director of the project.

Research Program

The research program as established to date has been divided into three phases designated as Test Programs #1, #2, and #3.

Test Program #1 has been completed, and a preliminary report has been published. The purpose of this program was two-fold: (1) to perform preliminary, investigatory tests to establish successful methods of operation for the tests to follow, (2) to perform certain basic investigations which were important in themselves. There were three series of tests conducted in this program:

1. Influence Lines for Direct Stress Determined Experimentally.
2. Stresses and Forces with One Track Loaded.
3. Stresses and Forces Under Full Design Load, and the Establishment of a Loading and Testing Procedure for the Destructive Test of the Endposts.

The purpose of Test Program #2 is to establish a method of estimating the ultimate load of damaged endposts. This problem arises when a train load has shifted and struck and bent an endpost. This program has been divided into 2 series:

1. Destruction test of straight endposts.
2. Destruction test of bent endposts.

Determination of the effects of the floor system in truss action and of the stresses introduced by the erection of the floor system are the objectives of Test Program #3. This program has been divided into three series:

1. Stresses in Loaded Bridge with Floor System Removed.
2. Stresses in Loaded Bridge with Floor System in Place.
3. Stresses Introduced During Erection as a Result of Clearances Between the Stringer and the Floor Beams.

Tests that have been run so far have themselves suggested many new areas of investigation. Definite plans have not been made beyond those mentioned above, because analysis of the data from these tests may suggest a new, more interesting, and more rewarding group of tests. In all past tests slip gage readings at the joints were taken, and will be taken on all future tests. These readings indicated that no slip had occurred.

Current Results

The general conclusions and a general discussion will now be given for each of the completed tests.

Influence Line Tests

The influence line tests indicated that the forces computed on the basis of the simple truss assumption are very close to those found experimentally. The only exception to this occurs in the lower chord and diagonal members, where the measured stresses were lower than those computed. The reason for the lower measured stresses is, of course, that the floor system acts as part of the truss's lower chord. The precise effects of this action will be investigated further in Test Program #3. To summarize, the conclusions reached on Test Program #1, Series I, were:

1. The forces computed for the upper chord using the simple truss assumption are within 5 percent of those found experimentally.
2. The measured force in the lower chord and diagonals was between 62 and 76 percent of the computed stresses.

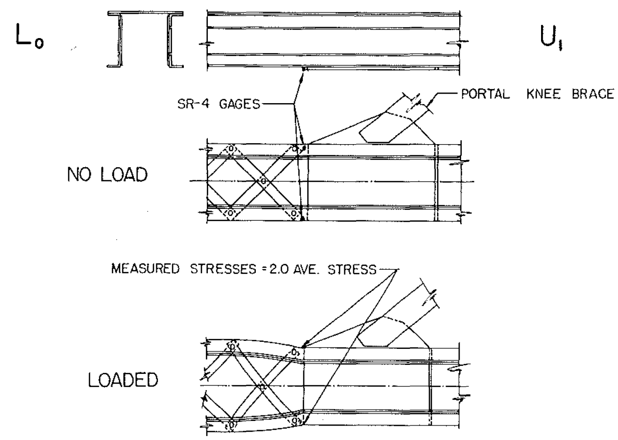
Stresses and Forces Under One Track Loaded

There are two purposes of this series: (1) to determine the distribution of forces to the two trusses with one track loaded, and (2) to determine the distribution of stresses within the members of each truss with one track loaded. The force in a given member with one track loaded was between 60 and 78 per cent of the force with two tracks loaded. This indicates that as to be expected the forces are less when only one track is loaded; however, the secondary stresses that occur under the eccentric loading must also be considered. In several sections the stresses were higher with one track loaded than with two tracks loaded.

Stresses and Forces in Dry Run Test

This test served as a "dry run" for the destructive test of the endposts which was to follow. During this test, the loading system, the required instrumentation, and the fact that it would be possible to fail the endposts without damaging the high strength members of the bridge were established. In addition several results were discovered that were interesting in themselves.

1. (Fig. 3) The stay plates on the endpost of Truss A appeared to cause stress concentrations in the outstanding legs of the angles outside the stay plate connection. The concentration amounted to 2.0 times the average measured stress in the member.



MEASURED STRESSES AT END OF STAY PLATE

FIGURE 3

It is believed that this concentration was caused by two things:

a. The pantographic action of the lacing bars tends to "spread" the lower angles when the member is compressed, and, as can be seen in Figure 3, this will introduce compressive bending stresses at the toes of these angles since the stay plate does not permit such spreading.
b. The gusset or stay plate, is quite long, and therefore tends to act as a coverplate, picking up a certain load. This load must be transferred from this plate, and the only connection to transfer this load is to the outstanding legs of the lower angles, thus causing a stress concentration at this point.

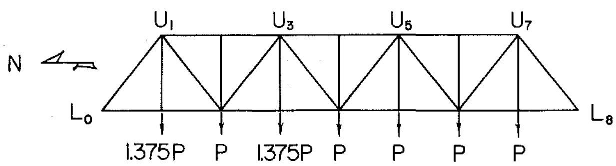
2. The stresses at the L_0 joint were subject to considerable stress concentrations, and (excluding the concentrations caused by the ovaloid perforations) the stress was 1.7 times the average measured stress in the member.
3. The stress concentration at the edges of the ovaloid perforations on Truss B was tabulated, and the overall average value was 1.67 the average measured stress.

Destructive Test of Straight Endposts

This test was conducted on November 4, 1959, and has received considerable publicity since then, although no data has been released except for the failure load. Much of the data has been reduced, but not all of it has yet been "analyzed." The term "analyzed" is used here to mean "an endeavor to make the data meaningful." Some tentative conclusions have been reached already and checks to determine the validity of these conclusions are being performed. Before discussing the conclusions, the loading and instrumentation will be briefly explained.

Loading

The loading found to be satisfactory is shown in Figure 4. This corresponds to two trains, both traveling north, and positioned for maximum stress in the endpost. Naturally, the loading could not be "exact," since there are only two different loads other than zero that can be supplied by the hydraulic system. The load was idealized, keeping the stress in the endpost the same.



LOADING

FIGURE 4

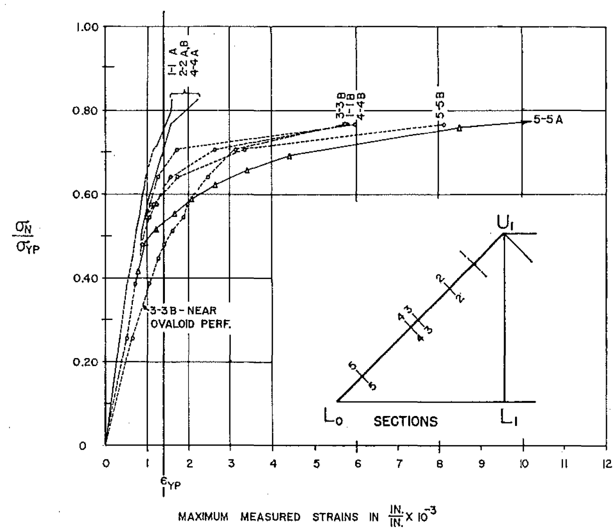
Instrumentation

Three hundred SR-4 strain gages were used to measure the strains in the members. The deformations, both linear and angular, were measured by means of level readings on scales that were attached to thin, hanging weighted wires. Although rather primitive equipment was used, these readings produced useful and fairly accurate data. The new, very precise, optical system which will greatly improve the accuracy of displacement data, will be in place for the next test. For our next test we will use these measured displacements and rotations to determine the moments and direct forces acting on the ends of the member, as the SR-4 data showed the member subject to considerable concentrations and distortions near the gussets. The SR-4 data will serve as a check of the optical data and vice-versa.

Conclusions

It is to be understood that the conclusions given here are, for the most part, tentative and are based on only a partial analysis of the data. This will then be considered as a progress report of this test.

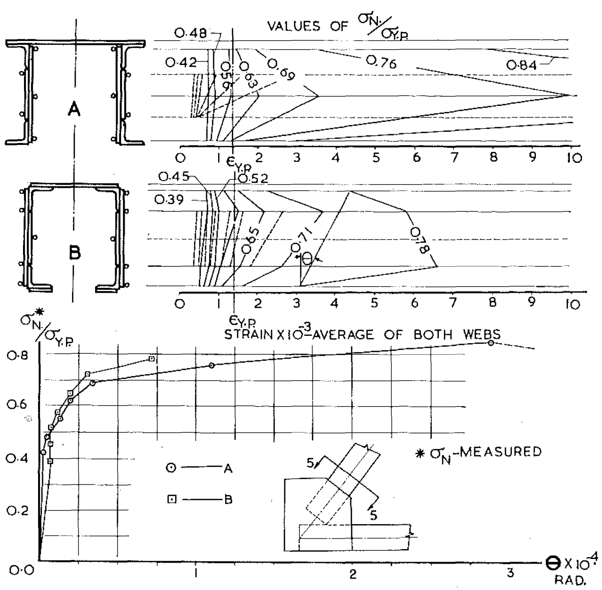
1. The ultimate load was 2.3 times design load of the end post; where the design load includes live load, dead load, and impact.
2. Both of the endposts failed! Proof of this is demonstrated by the facts that (1) the bridge continued to deflect under a constant load, and (2) when a limit switch was tripped, the load began to decrease gradually, and under this decreasing load the measured strains in both endposts continued to increase. In addition, several plots made during the test showed that the load vs. deformation curve was flat at the ultimate load.
3. SR-4 strain gage readings indicated that at several locations there occurred, either a local buckle, or a local bend in one of the elements of the main material or gusset plate. Our subsequent tests will measure these local deformations to determine their character in more detail.
4. It appears that the failure began at the L_0 joint of each truss. Figure 5 shows the increase of maximum measured strains at each of the sections along the length of the member for both trusses. Note that the strains at section 5-5 are greater than the strains at any other section for both trusses. Note that there is a great difference between the strains at 5-5 and the other sections for truss A. This indicates that the ultimate load could be increased by improving the design of this joint on the A truss.



MAXIMUM MEASURED STRAINS

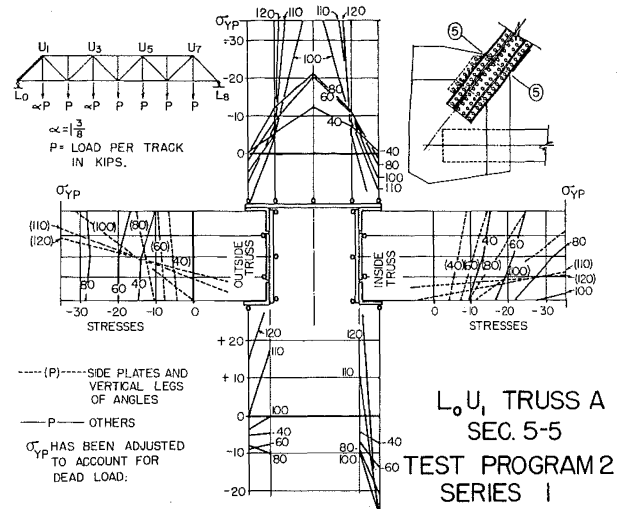
FIGURE 5

On truss B, however, the strains are greater at 5-5 than the other sections, but it appears that this endpost failed in a more "integral" fashion than did truss A. This is borne out by the fact that a whitewash coating on truss B showed cracks joining the rivets along the complete length of the member. This section, then, should be investigated more closely. Figure 6 shows the measured strains in the webs and also the measured strains in the side plates and vertical legs of the angles. These values are the average for both sides of the member. They are plotted for various measured values of



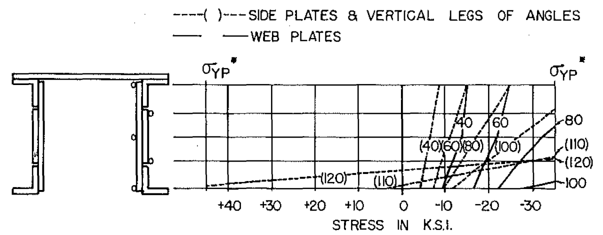
PLOT OF ROTATION AND STRAINS AT L₀JOINT

FIGURE 6



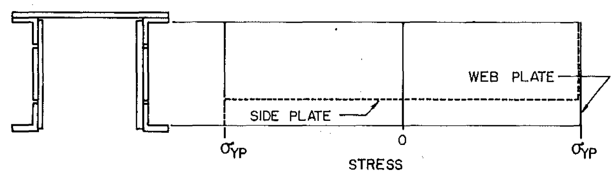
MEASURED STRESSES

FIGURE 7



ACTUAL MEASURED DISTRIBUTION

* σ_{YP} HAS BEEN ADJUSTED TO ACCOUNT FOR DEAD LOAD.



IDEALIZED DISTRIBUTION AT FAILURE

STRESS DISTRIBUTION

FIGURE 8

σ_{normal}, divided by σ_{yp}. σ_n is for live load only, and $\frac{\sigma_{DL}}{\sigma_{yp}} = 0.12$ for truss A and 0.11 for truss B. There are several interesting facts to observe from these plots:

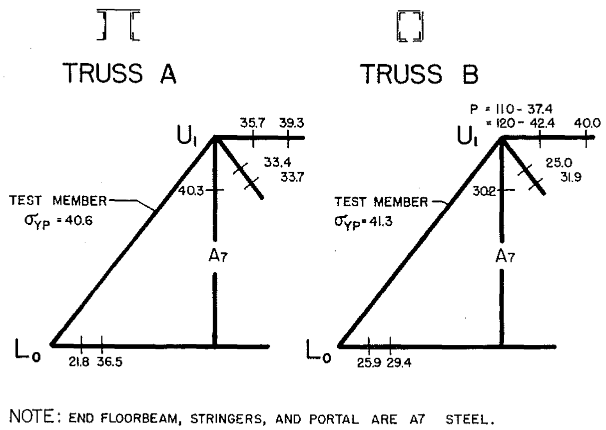
(1) The concentration of strains in the center of the web indicates that the distribution of rivets in the connection could be improved to give a more efficient distribution.

(2) The "rotation of the plane of strain" is plotted and indicates that the member did fail at this section.

Figure 7 shows the measured distribution of stresses at this joint in Truss A. Note that the dotted lines which represent the stresses in the side plates and vertical legs of the angles lag behind the solid lines which represent the stresses in the webs. Note also that the character of the distribution is quite different for the solid and dotted lines. The stress data will be presented in such form as is shown here in our next report. Figure 8 shows an enlarged view of the stress distribution shown in Figure 7. Note that the webs have a completely different distribution than the side plates and vertical legs of the angles. This is due to the influence of the gusset as a stress-raiser, causing the entire web to be at yield point stress in compression, whereas the side plates and vertical legs of the angles are still partially elastic. The web portion of the section then can not withstand any moment. The side plates and vertical legs of the angles supply the moment resisting capacity and the section is being analyzed on this basis. This distribution of moment and direct load resistance in the section is true only for this particular ratio of moment and direct load. The ultimate load of this test is in close agreement with our analytical study using the experimental distribution. We hope to publish this as soon as we have more experimental verification.

5. It is thought that a redesign of this joint, especially for Truss A, would be beneficial, and that the ultimate load could be increased since it was the L_0 joints which precipitated the failure. In our next series of tests we will devote more attention to studying the action of these joints and thus we hope to be in a position to offer recommendations

for improvement of the design details. It is interesting to note that this type of failure could not have been realistically achieved in the normal testing machine test. This fact confirms the foresight of Prof. Parcel in his original realization of the need for a model test bridge. In connection with this point I would like to answer the question, "How accurately were the end conditions of an all A7 bridge reproduced in the model bridge?" Let us examine Figure 9. The purpose of this test was to determine the ultimate load of the endposts; therefore we must not allow the upper chord, for example, to fail, as this would defeat the purpose of the test. For this reason the upper chord was made of medium manganese steel.



MAXIMUM MEASURED STRESSES IN MEMBERS ADJACENT TO TEST MEMBER

FIGURE 9

U_1 Joint: The maximum measured stresses in the various members at the U_1 joint are shown in Figure 9, and it will be noticed that these stresses are less than the yield point of A7 steel. When the dead load stresses are added to these they do not exceed the yield point of A7 steel until the final increment of load. Therefore, aside from a buckling failure of the upper chord, this joint performed as if it were made up of A7 members up to the final increment of load.

L_0 Joint: The same is true at the L_0 joint probably because of the action of the floor system as part of the truss. We can conclude, therefore, that the end conditions, aside from a buckling of the upper chord, were as they would have been in an all A7 bridge up to the final increment of load.

6. The stresses at failure are shown in Figure 10 for each of the endposts. The calculated stress is again that stress computed using the simple truss assumption.

MEAS. L.L.	COMPUTED L.L.	COMPUTED D.L.	TOTAL MEAS.	TOTAL COMPUTED	YIELD POINT	ULTIMATE / YIELD POINT	
35.2	33.8	4.85	40.0	38.6	40.6	0.98	TRUSS A
32.1	32.1	4.60	36.7	36.7	41.3	0.89	TRUSS B

STRESSES IN L_0U_1 AT ULTIMATE LOAD

FIGURE 10

7. We can now consider a comparison of the actual test results and what we might have expected. It has been fairly well established in testing machines using columns with flat or pinned ends, and having an $\frac{l}{r} = 57$, that the column's efficiency, $\frac{\sigma_{ULT}}{\sigma_{yp}}$, is between 0.85 and 1.00, depending upon the cross-sectional shape of the member. There are several factors which would operate to lower the values which this would lead us to expect. These are:
   (1) The endpost did not buckle about its strong axis, nor in a plane of symmetry, nor was it free from twisting.
   (2) The failure was actually one which began in the joint, not permitting the column to attain its full capacity.
   (3) End moments introduced shears which some engineers have suggested would reduce the column efficiency and possibly cause a sudden collapse.

The only factor which had a tendency to increase the column efficiency was the reverse curvature to which the end post was subjected.

The net result is that we would expect the value of the column efficiency to be considerably lower than the 0.85 to 1.00 which testing-machine tests have yielded.

What we found, however, was a column efficiency of 0.98 and 0.89, which is even greater than 0.85; and, certainly, no sudden collapse occurred. This is quite reassuring to all concerned with the ultimate capacity of a column under actual end conditions.

Future Plans

DAMAGED ENDPOST TESTS

The objective of the damaged endpost tests is to establish a means of quickly estimating the capacity of damaged endposts. This program of investigation divides itself into two separate phases: (1) experimentally finding the capacity of damaged endposts under controlled conditions, and (2) predicting the capacity of damaged endposts in the field based on these experiments.

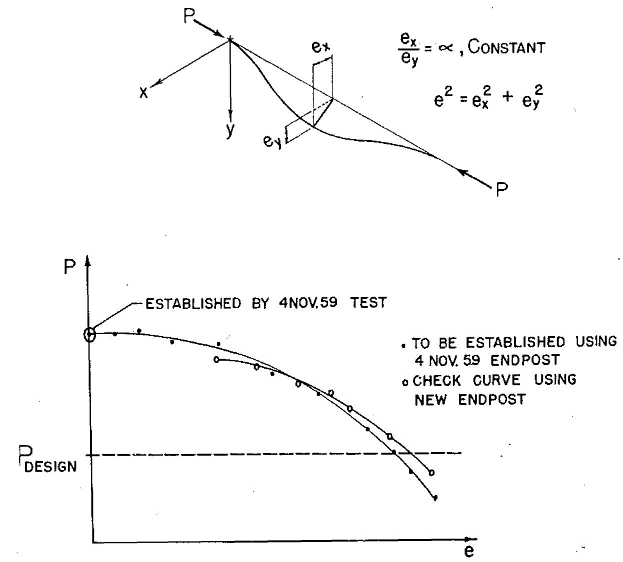
Phase (1) will be carried out this summer and early fall. We will establish a curve of ultimate load vs. initial bend (Fig. 11) by introducing a small bend, say e_1 in the existing straight endpost and then loading to failure. Following this, the bend will be increased to e_2 and the endpost again loaded to failure; and so on, until our curve has been established. Since this procedure raises the question as to how this repeated bending and failing affects the "ultimate load vs. initial bend" curve, we will answer the question in the following way:

> We will perform small-scale laboratory tests under "ideal" conditions on rectangular bars, establishing the P vs. e curve, using the progressive bending and failing routine. Then the P vs. e curve will be established for specimens not previously failed. A comparison will answer the question.

Since there is always a scatter of experimental data, we will attempt to duplicate our results of the endpost test in a series of similar tests using two other endposts which are now on hand for this purpose.

Our first destructive test showed the important influence of joint details on the ultimate load of the column, and for this reason a more detailed study will be given to the joints during these subsequent tests.

The number of SR-4 gages will be about doubled, stress coat or photo stress will be used on the joints, optical data will be taken to determine the rotations and displacements, and dial gages will be installed to study, among other things, the local deformations.



PLAN FOR THE BENT ENDPOST TEST

FIGURE 11

Test Program #3

INVESTIGATION OF FLOOR SYSTEM

The objectives of this test program are:

(1) To determine the participation of the floor system in truss action.

(2) To determine the erection stresses, which are caused by the end clearance between the stringer and the floor beam.

This program will be carried out immediately after completing the testing of the damaged endposts.

Resume

We believe that one fact has now definitely been established. We now have a facility to check new design concepts, to verify new analytical theories on a real bridge, not only in the elastic range, but up to and including ultimate loads.

A Research Development Goes to Work

MACE H. BELL

Promoting and advancing more economical, more efficient steel construction through research, development, and advisory engineering programs is one of the most important functions of the American Institute of Steel Construction. The Institute is constantly engaged in research activities, and in the past few years has greatly expanded this program.

A large part of the overall research program is carried on through cooperation with other interested organizations and governmental bodies as co-sponsors of projects guided by Research Councils. The Institute also sponsors several research programs which it alone finances and controls. In addition, it makes Grants-in-Aid to various colleges and universities to assist student and college professors in carrying out projects of limited size and scope.

It is from these various sources that new concepts and developments, as well as improvements in existing practices, are born. As an alert and progressive industry we must determine from the large amount of information obtained, those ideas and practices which are practical, economical, and useful. Then comes the task of turning these ideas into practical, useful tools which will improve and extend the use of structural steel. Here is where education, promotion, and selling come into play to put a research development to work.

In talking to you a year ago, I spoke of five broad fields of work, in which the area engineers concentrate their efforts. Today, I want to tell you about one outstanding research development, Plastic Design in Steel, and summarize for you the part our area engineers have played in advancing this completely new design concept, from the laboratories of Lehigh University to practical application in many projects throughout the country.

A Brief History of the Research Program

For many years, structural engineers have sought a rational explanation of behavior of continuous steel frameworks when subjected to overloads. It had long been known that such structures, when subjected to severe overloadings, foundation settlements, and even destruction of intermediate supports, still gave good accounts of themselves under these unforeseen conditions. But, elastic design indicated they would fail. What was the answer?

History tells us that the first conscious application of the principles of plastic design was used in an apartment building constructed in Hungary in 1914. For nearly a quarter of a century the "idea" lay dormant except for limited study and research on the part of a few designing engineers and researchers. Then, with catastrophic bomb damage raining from the skies in World War II, plastic design saw considerable practical and successful application in 1939 in the design of steel shelters to protect British families.

In 1946, following the close of World War II, an extensive research program on plastic design began at Lehigh University. The program was co-sponsored by the Welding Research Council, the United States Navy Department, AISI, and AISC. This program undertook an extensive and thorough study of the ultimate strength of welded continuous frames and their components. Almost from the very start, the research program began to pay dividends by adding to our store of knowledge regarding the real behavior of the component parts of a structure. Many questions related to the design of welded corner connection of rigid frames, amount of lateral support required, effects of residual stress and so on, were answered, and reports were made available to the engineering profession. It was at this early date, that a partial recognition of the reserve plastic strength of continuous frameworks was started in this country by the introduction of 20 percent increases in allowable stress in the AISC specification of 1946.

As the work of the research team at Lehigh progressed, papers and discussions on the test results and the early development of design theories were published in both the Research Supplement of the Welding Journal, and in the Proceedings of the ASCE. Many of you will recall Dr. Lynn Beedle's report on the research findings at the 1952 National Engineering Conference, followed by his second report as a part of a symposium on Ultimate Strength as the Design Criterion at the 1954 Engineering Conference.

By 1955, the theory of Plastic Design In Steel had nearly come of age for one- and two-story rigid frame type buildings. At this point the research team was not yet in a position to promulgate complete design rules, but they were able to offer promising glimpses of the ease of design and of the substantial savings in steel weight which could be realized by utilizing the new design method.

Acquainting Educators and Engineers With Plastic Design

In September of 1955, a nine day Summer Course, sponsored by the AISC and Lehigh University in cooperation with the American Society of Engineering Education and the Structural and Engineering Mechanics divisions of the ASCE, was held at Lehigh University. This was the first step in introducing these new design procedures to educators, to designing engineers, and to engineers

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associated with our industry. Out of an attendance of some three hundred, 131 faculty members from 81 universities were present. One result of this conference was that, today, a great many universities and colleges include plastic design in their curriculum. Another result was that within a relatively short time some forward-looking designers began to use the method in one- and two-story frameworks.

As a follow-up to this national introduction to the completely new design concept, copies of the Lehigh Summer Course lecture notes, and test demonstration reports, were sent to each of our area engineers in the field. Here, for the first time, the area engineers began to actively promote meetings, seminars and symposiums devoted entirely to discussions of the research findings and the development of design theories. Beginning in late 1955 and continuing through mid-1958, over 50 seminars of this type were held. Some took the form of one or two day conferences, others consisted of a series of weekly lectures. One or more members of the Lehigh team who guided the research, together with Ted Higgins and Ed Estes, and faculty members at many of the universities made up the lecture teams. These seminars, arranged by the area engineers in cooperation with universities and fabricators all over the United States, served to acquaint practicing designers, educators, and students with the many advantages to be realized from plastic design. Although no accurate records of attendance were kept, it is estimated that the total attendance was well over 3,000.

The next educational step, taken in the same period as the seminars, was to acquaint the engineers of the Institute's member fabricators, and their guests, with this new design tool. This was accomplished by devoting the entire three day program of the 1956 National Engineering Conference to Plastic Design In Steel.

Also in 1956, the Committee on Plasticity Related to Design took action to join with an existing Welding Research Council Committee to prepare a "Commentary on Plastic Design." First drafts of the commentary were written by research-staff members at Lehigh University. As the various chapters of this commentary appeared in the Journal of the Engineering Mechanics Division of ASCE, they have been reprinted and distributed by direct mail, and by the area engineers, to engineers, educators, and building officials. These were published in 1959 and 1960.

Research and development of theory had reached the point where a specification and design manual could be written in 1958. On December 4, 1958 the first plastic design specification in the United States was issued by the AISC. This was followed, in January 1959, by issuance of the AISC Manual—Plastic Design in Steel. These furnish the tools needed by the practical designer in everyday practice.

Shortly after the PD manual became available, plans were started for a series of lectures on the practical application of this new design method. At our National Engineering Conference in Birmingham, a year ago, the AISC Board of Directors authorized special funds to prepare material for these lectures, and to retain professors or consulting engineers to present them to architects, engineers, educators, and building officials.

A team composed of Mr. W. A. Milek, District Engineer at Omaha, Ted Higgins and Ed Estes was formed

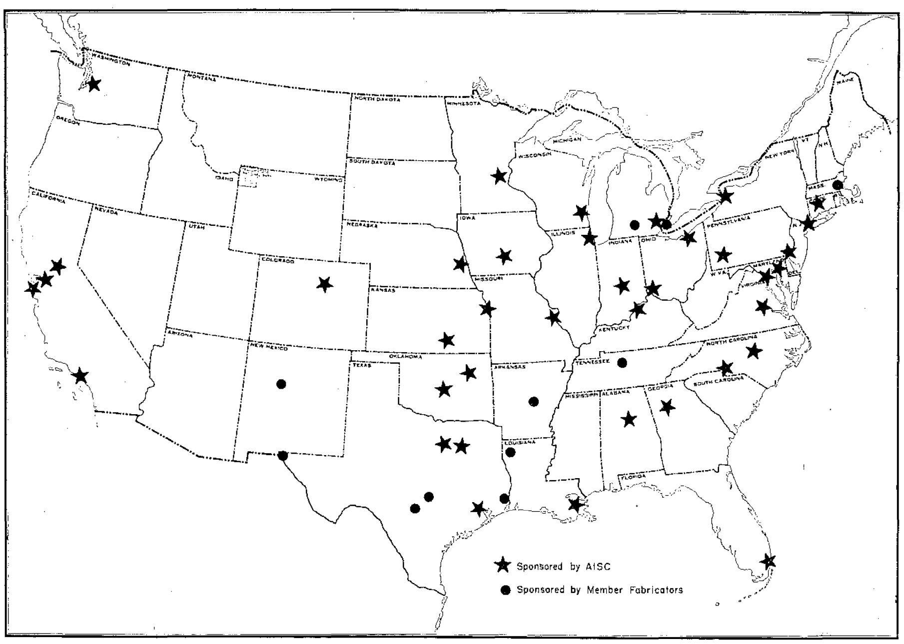

FIGURE 1

in New York to prepare lecture outlines, select suitable design examples, and develop solutions for each problem. High praise is due the outstanding work performed by this team which, working under pressure, completed notes for six two-hour lectures with drawings, sketches, and computation sheets, all illustrated by 174 color slides, in time for the 1959 Summer Series.

Meanwhile, the area engineers involved completed all arrangements for lectures in their areas. Then as the lectures began, they, with the assistance of many individuals from member companies, handled all of the arrangements necessary to insure their success.

These practical application lectures on plastic design have been accorded an enthusiastic reception throughout the country. They, without doubt, represent one of the most outstanding promotional and educational programs ever conducted by the Institute. Figure 1 shows that lectures were held in 38 cities located in 27 states, crisscrossing the nation from New York to Los Angeles, and Seattle to Miami. Total attendance in the 38 cities was 7,000.

In the same ten month period, groups of fabricators in several areas of the country sponsored and financed similar lectures with the assistance of AISC. These special series were conducted in eleven different cities in seven states, and drew an approximate attendance of 1,000.

This educational program, designed to acquaint practicing engineers with the supplementary rules, the contents of the PD manual, and the many advantages to be gained by plastic design will continue for some time in the future. Schedules are now being arranged for lectures in several additional cities. Basic criteria used in designating locations include availability of a qualified lecturer, anticipated attendance of approximately 100 persons, availability of suitable lecture hall facilities, and an offer on the part of fabricators in the area to assist in organizing and conducting the series.

Figure 2, prepared recently by Dr. Beedle, pictures in

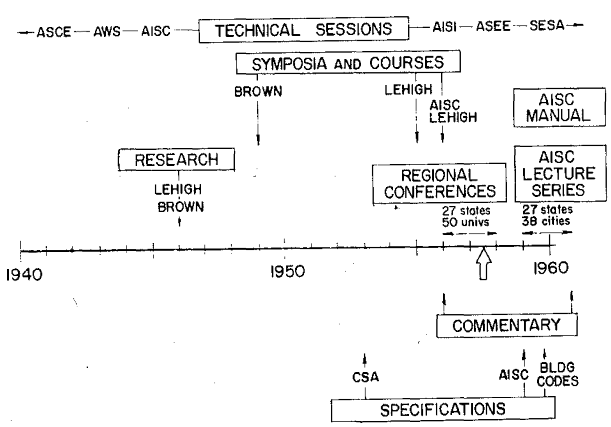

FIGURE 2

capsule form the highlights of this brief 15 year history beginning with the start of research in 1946, and continuing right up to the current lectures and building code work of 1960.

Building Code Acceptance of Plastic Design

With adoption of the Supplementary Rules and publication of the AISC plastic design manual, the way was opened for acceptance of the new design method by building officials. Written requests were submitted to regional code authorities, to state building code authorities, and to building code officials of many of the larger cities, requesting that the Supplementary Rules be approved and incorporated in building codes under their jurisdiction. We have had no difficulty in obtaining an administrative ruling approving the use of plastic design in almost every instance. Nevertheless, we have taken these formal steps to obtain code approvals just as we did several years ago in the case of the high strength bolt specifications. The area engineers have been in the forefront of this work preparing formal requests, and meeting with building code officials and Code Change Committees in a stepped up campaign to secure early adoption throughout the nation.

In the 15 month period since this program was undertaken, building officials have been quick to recognize the advantages of this new design technique, and the thoroughness of the research and development program which documents the rules. As a result, the code change machinery of various authorities was put into motion in many areas, and committees undertook the task of incorporating the rules into codes. In some cases this has been done by adopting the rules as a standard. In others, each provision of the rules has been written into the appropriate sections of existing codes.

First, as many of you know, there are four so called "nation-wide" building codes in existence in this country. These are the Basic Building Code of the Building Officials Conference of America, the Uniform Building Code of the International Building Officials Conference, the Southern Standard Building Code of the Southern Building Code Congress and the National Building Code of the National Board of Fire Underwriters. Plastic design in steel is now adopted in the first three of these codes, and we are advised by the National Board of Fire Underwriters that they accept the supplementary rules under the provision of their building code referring to the AISC specification as a recognized standard.

These four model codes have either been adopted, or used as a guide in drafting building codes, by several thousand cities throughout the country. The exact number of such cities is unknown, but estimates place it at between 2,500 and 3,000. As each of these cities and towns revises its codes to conform to the latest edition of one of these model codes, provisions for plastic design will be written into an ever increasing number of city codes. Meanwhile, in the interim period, many of these cities will permit the use of the new design method on strength of the fact that these national authorities have approved it.

A survey begun in March of this year and still continuing shows that 78 cities of 100,000 and greater population have indicated they will approve the use of plastic design, and that 17 of these have adopted code provisions on plastic design.

The survey covered all cities of over 100,000 population, of which there are 137 in the United States based on 1958 estimates. The survey also covered states in which there are building codes, and federal authorities which promulgate codes and specifications applicable to work under their jurisdiction.

We have reports that five states—New York, North Carolina, Ohio, Indiana, and Wisconsin—all have incorporated provisions in their statewide codes which permit plastic design.

With regard to federal agencies, their procedures permit the use of plastic analysis for protective construction. Up to the present time regulations contained in the Engineer Manual of the Department of the Army, Chief of Engineers, in the Standards of Design of the Department of the Navy, Bureau of Yards and Docks, and in the Structural Engineering Handbook of the General Services Administration, Public Buildings Service, do not specifically provide for its use in ordinary construction. In each case however, it has been indicated that anticipated revisions currently underway in these various standards will permit further use of plastic design.

Considering the fact that formal adoption of new specification material in building codes takes considerable time, the progress that has been made to date in this very important area is most gratifying. By way of comparison, a period of four years, from 1951 to 1955, was required to secure widespread adoption of the specifications for high strength bolting.

Application of the Method in Design

A chart of Dr. Beedle's (Fig. 3) shows the number of structures known to have been built in the United

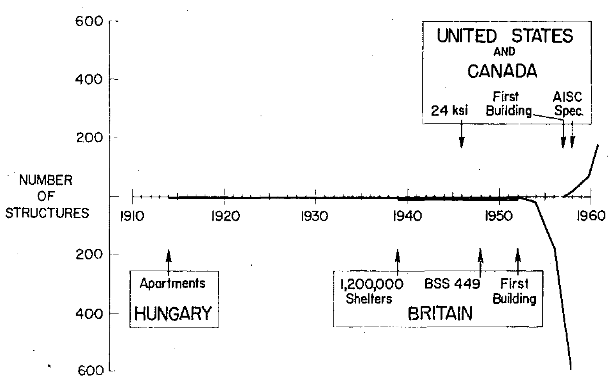

FIGURE 3

States and Canada as ordinates to the curve above the calendar line, and the number of structures built in Britain as ordinates to the curve below the line. It is clear that the number of known plastically designed structures in the United States, about 175, has not begun to approach the number built in Great Britain, which had reached approximately 600 in 1958 as shown on the chart, and has now reached an estimated 1,000 structures. However, the rate of progress, during the early years since the first structure was built in 1957, seems to compare favorably with that reached in Great Britain in a similar elapsed time.

The diagrammatic sketches shown in Figure 4 indicate the types of building frames that have been built. They cover continuous beams on columns, gabled frames (both single and multiple span), two-story structures, multistory braced frames, and several frames with somewhat unusual geometry. At the present time, in the United States, the method is most commonly employed in continuous beam type spans, and in rectangular and gabled single-span frames.

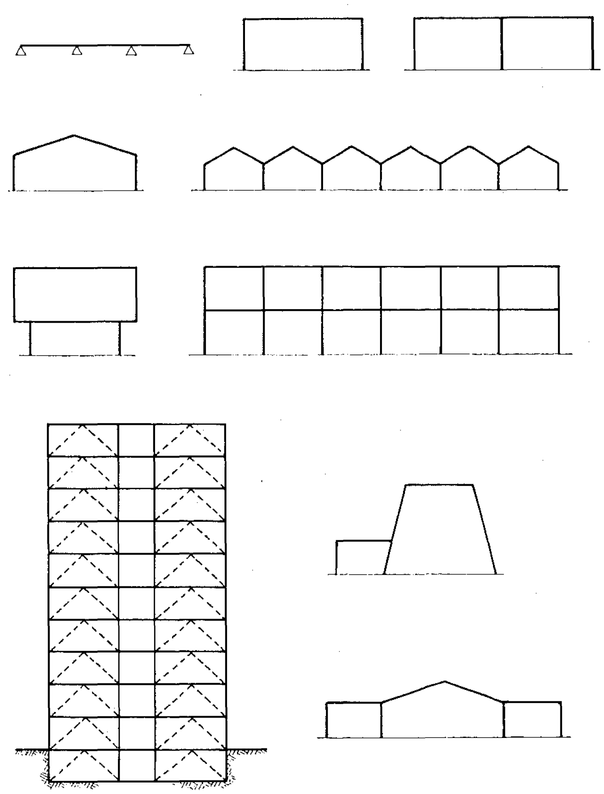

FIGURE 4

One very interesting feature, brought out by a recent survey conducted by Dr. Lynn Beedle, has been the function for which plastically designed structures have been erected. Early predictions indicated that warehouses, military structures, and industrial buildings would account for a large number of applications of this tech-

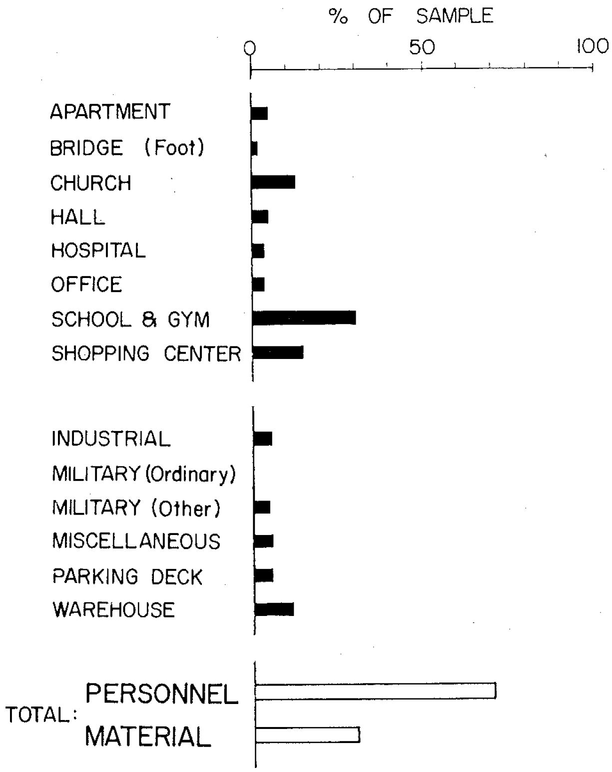

FIGURE 5

nique. Dr. Beedle's survey indicates that quite the reverse trend is true, as is shown in this graphic presentation (Fig. 5). Note at the bottom of the figure, that the majority of applications, (70%) are for housing personnel, and that structures for storage and a handling of material account for only 30% of the total. The top part of the graph indicates that schools and gymnasiums (30%), shopping centers (14%), and churches (12%), account for more than three-fourths of the total applications classified as being for personnel at the bottom of the chart.

In an earlier survey made in November, 1959, about 75 known plastically designed structures had been reported as erected in 13 states. The present estimate, from which Figure 5 was constructed in March, 1960, indicates that 175 structures in 27 different states have been individually designed and erected.

The first such known structure built on the North American continent is a two-story office building in Kingston, Ontario. According to Architectural Forum and Engineering News-Record reports, savings in weight of steel for this job over conventional design amounted to 22%. (See Steel Construction Digest, 2nd Quarter, 1957.)

The first plastically designed building in the United States was an 88-ft. span rigid frame warehouse for the Dalton Company, located in Sioux Falls, South Dakota. Civil Engineering magazine described this structure in its September, 1957 issue.

Space does not permit a review of the many types and sizes of plastically designed buildings which have recently been constructed in every section of the country. However, I would like to point out a few specific projects to illustrate some of the things that are being done in this field.

Three plastically designed buildings were recently completed for the Safeway Distribution Center, Omaha, Nebraska. The larger building, 473-ft. x 450-ft., utilized continuous beams on columns. Plastic design achieved a 14% weight saving, with a cost saving of $24,500. This building, designed by Leo A. Daly Company, was reviewed in the Steel Construction Digest, 3rd Quarter, 1959.

The new plant of the Gates Rubber Company, Nashville, Tenn., constitutes one of the largest structures in area that has been plastically designed in the United States. (See Figures 6 & 7.) This warehouse and manufacturing plant, designed by the Rust Engineering Company in Birmingham, consists of six continuous gabled frames with spans of 60 feet each. Column heights are 21 feet and the frames are spaced on 25-ft. centers. The 360-ft. x 975-ft. building has a floor area of about

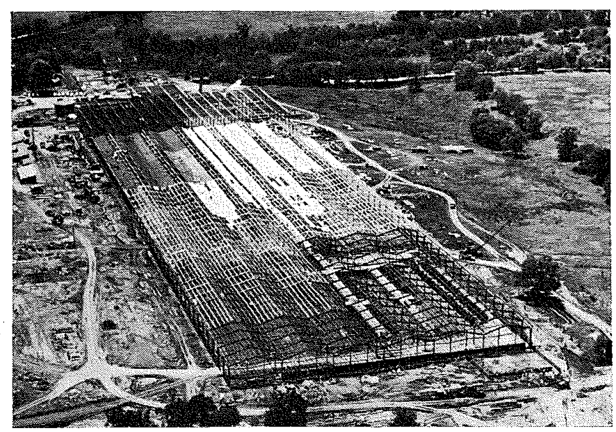

FIGURE 6

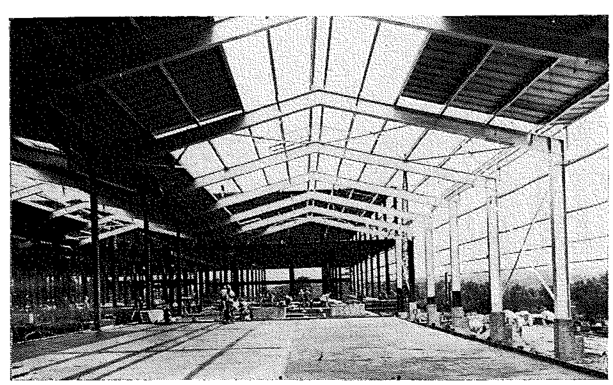

FIGURE 7

350,000 sq. ft. A weight saving of 19% (140 tons) was achieved in comparison to conventional elastic design. This structure is described in Steel Construction Digest, 3rd Quarter, 1960.

The Museum Building of the Metropolitan Boston Art Commission (Fig. 8 & Fig. 9) is a two-story structure in Boston designed by plastic methods because the engineers, Goldberg and Le Messurier of Boston, felt that it was the only logical solution that would afford protection in the event of small differential settlements in the foundations, placed on the edge of a tributary of the Charles River.

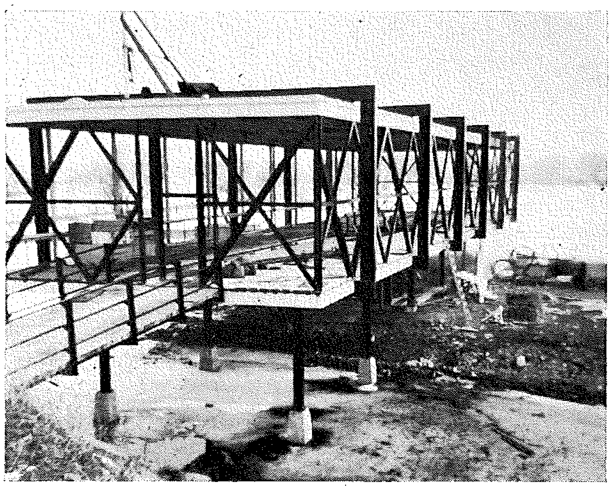

FIGURE 8

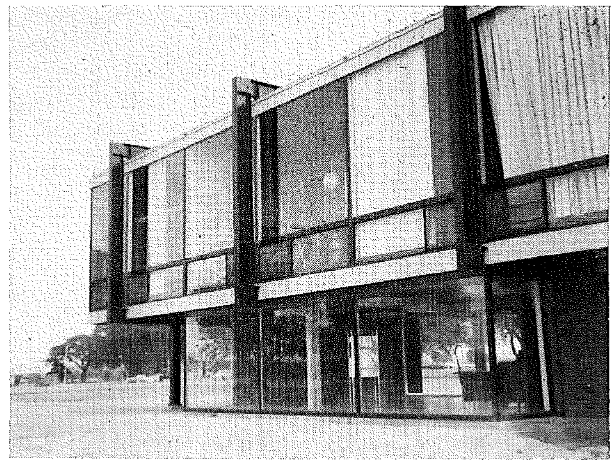

FIGURE 9

And now, my story of how a research development is being put to work is told. In the construction industry, designers and owners are always open to alert, aggressive selling combined with imagination and new ideas. Plastic design is a completely new idea. It offers: a saving of material; a saving of design time, a completely rational design basis; and means of investigating many design possibilities in a short time. We have given the designers the tool they need to use the new method. It is based upon properties peculiar to structural steel alone and can not be imitated with other structural materials. We are encouraging them to use it all over the country!

A Higher Yield Point for Structural Steel?

T. R. HIGGINS

In a letter to the Chairman of ASTM Committee A-1 on January 25, 1957, Mr. R. W. Binder, Chairman of the AISC Committee on Specifications, phrased it this way:

"The Committee would like to have your opinion whether it would be practical to raise the minimum specified yield point called for A7 steel to 39,000 psi, and if not practical how much it might be raised."

For some time prior to this the AISC Committee on Specifications had been exploring various possibilities for liberalizing its working stress recommendations. Obviously, one such was that of raising the minimum strength requirement under which plain material is furnished. This would require favorable ASTM action.

At the same time in a letter to the American Iron & Steel Institute Mr. Binder asked for information on the following points:

1. How much difference is there in the minimum and average yield point stress of structural steel as furnished today under ASTM Specification A7 and as produced in 1934?
2. How much of an increase in minimum yield point would you consider to be practical?
3. What proportion of structural steel is currently produced under ASTM Specification A7 as compared with A373?
4. The effect of an increase in yield point upon the price of structural steel, if no limitation were placed upon carbon and manganese.
5. The increase in carbon and manganese (over and above that presently specified in A373) that might have to be added if the guaranteed minimum yield point for A7 were increased approximately 20% above the present 33,000 psi.

Before attempting to follow the vicissitudes of Mr. Binder's queries it might be well to put a scale on the size and scope of the specification-writing activities of the American Society for Testing Materials. We in the structural steel fabricating industry are rather in the habit of thinking of ASTM only as the authority for specifications covering three or four grades of structural steel, supplemented by a few standards for rivets and bolts.

Actually, in the field of ferrous metals alone there are over 300 ASTM specifications, covering steel pipe, tubes, fittings, rails and many other steel products, in addition to the dozen or so specifications for the more familiar products passing through member company plants.

Mr. Higgins is Director of Engineering and Research, AISC, New York, N. Y.

Responsible for the contents of over 200 of these specifications is ASTM Committee A-1 on Steel, the oldest of all of the ASTM technical committees. It has a membership of roughly 350. Approximately 50 of these are classified as having a general interest. The rest of the committee is almost equally divided between producer and consumer representatives.

Obviously, the work of such a large committee must be taken care of largely in sub-groups. Altogether there are 17 standing sub-committees under A-1, of which Sub-Committee II is charged with the responsibility for specifications dealing with structural steel for bridges, buildings, rolling stock and ships. The current membership of this smaller group is slightly over 50.

It was to Sub-Committee II that the January 25th letter was referred by the Chairman of Committee A-1. Under date of February 21st, copies of this letter were mailed to its membership for comment and a discussion of the problem was included in the agenda for the June, 1957 meeting of the Sub-Committee.

Written comments received prior to the meeting were largely negative. It is interesting to note that all of these were from consumer representatives (principally highway and railroad engineers) or from those listed as having a general interest (consulting engineers and inspection agencies). None of them came from producer representatives.

However, the reply received from the AISI Technical Committee on Plates and Shapes, prior to Sub-Committee II's June meeting, did not lend too much encouragement. First off, it stated that there was no significant difference between the minimum and average yield point stress of structural steel as furnished today and as produced in 1934 when the AISC Specification was revised to reflect an increase in specification minimum yield. This did not come as too much of a surprise but it did close the door on one possibility for liberalizing the AISC design specification.

The reply also expressed misgivings that the present specified minimum yield of 33,000 psi could be raised without some relaxation in elongation and bend test requirements and an increase in carbon and manganese. On the question of cost it could not make comment, referring this matter to determination by the several producing mills.

After considerable discussion of the whole subject at its June, 1957 meeting, Sub-Committee II adopted a motion placing itself on record with the opinion that "it is impractical to raise the present yield point with the present limitation on tensile strength, elongation and bend tests." The AISC Committee on Specifications was so notified and there the matter rested until the next annual meeting, when the subject was again brought up for informal discussion.

At this time the representative of AREA Committee 15 stated that that group would not increase its permissible stress provisions even if the yield point of A7 were raised. As a result again no action was taken.

When the subject came up for discussion at the Sub-Committee meeting last June it was finally voted that a Task Force be appointed by the chair to obtain firm commitments as to what action AASHO, AISC and AREA Committee XV would take in amending their standard specifications if the yield point of A7 were raised. If this group received favorable replies the Task Force was instructed to formulate a definite proposal for presentation at the January, 1960 meeting of Sub-Committee II. Chairman of the Task Force was Mr. M. A. Pinney, Engineer of Tests for the Pennsylvania Railroad Company.

In reply to Mr. Pinney's inquiry as to AISC's position and as to any reservations it would impose, the AISC Committee on Specifications stated:

> ". . . would favor an increase of 10 to 15 percent in the minimum specified yield point (now 33,000 psi). It would continue to use ASTM Specification A7 if the yield point were so increased and would adjust the basic working stress in the Institute's Specification accordingly.
>
> "The only reservations that the Committee would have are that the increase in tensile strength and minimum decrease in elongation not exceed the increase in minimum yield strength, and that any resulting economy in the use of the new specification would not be offset by an increase in cost of the material."

While Mr. Pinney was able to report less resistance to change on the part of other code writing bodies than heretofore, there still was considerable reluctance—notably on the part of the railroad car builders—who were fearful of losing some of the ductility associated with a 33,000 psi yield point. During these months of discussion, however, metallurgist representatives of the producing mills had been making every effort to reach a common agreement that would wrap up in one package the largest possible increase in yield stress consistent with the least possible change in ductility and cost.

As a result of the objections voiced by other users of A7 steel, it seemed best, when all of the provisions pertinent to a higher yield structural grade steel had been ironed out, to present it as a new ASTM Specification, merely referring to it as the ASTM Specification for Structural Steel and giving it a new serial number.

The final act of Mr. Pinney's Task Force was to recommend the presentation of this new Specification to Sub-Committee II for letter ballot.

Accordingly, copies of the draft were distributed in March of this year, with ballots returnable on or before April 27. Out of a total of 44 ballots cast only 5 were negative. And, after a conference in which minor revisions in the draft were agreed upon, all of these were withdrawn. Copies of the amended Specification were then distributed for letter ballot by the full membership of Committee A-1 prior to the June, 1960 meeting of that Committee to be held in Atlantic City.

Assuming that, with the unanimous endorsement of Sub-Committee II already in the record, A-1 approves the Specification at its meeting next month, the only remaining formality is its certification by the ASTM Committee on Administrative Standards 30 days thereafter.

Thus, barring the unforeseen, the action touched off by Mr. Binder's January, 1957, letter will soon have run its full course. But it would be unfair—yes, and ungrateful too—to stop there in giving credits. It is no secret that the goal which now appears to be in sight has long been a cherished dream of Mr. Duane Molthop, Chairman of this morning's session. To him let it be said, "Many thanks for the persistent prodding."

A detailed discussion of the provisions of the new Specification will be presented by Mr. John LeCron, Chairman of Sub-Committee II, whose patience and persistence has contributed so much to its promulgation.

A New Specification for Carbon Structural Steel

JOHN R. LeCRON

The preceding speaker has outlined the events leading up to the formulation of the proposed specification to be known as A36. It will be my intention to explain the important features of this document by comparing its individual requirements with those of ASTM Specifications A7 and A373. I am sure you are familiar with these two basic specifications.

All ASTM Specifications for Rolled Structural Steel follow a single format which includes a "Scope" clause as Paragraph 1. In this paragraph, the individual steel products which may be supplied are enumerated and the general fields of application are mentioned. Figure 1 shows the differences between the scope clauses of A7, A36 (the proposed specification) and A373. Using the A7 paragraph as a base, A36 has added only five words, which are italicized, to limit thickness. It is necessary to limit thickness so that the maximum chemistry can also be held to the lowest practical limits. Proceeding to the A373 scope, it can be seen that two additional words are present which indicate that this material is recommended for welded construction. Such a recommendation was also included in the original scope clause of A36 but was removed because of controversy with a minority group in the Sub-Committee II letter ballot. More will be said on this subject during a subsequent discussion of chemical requirements.

Mr. LeCron is Metallurgical Engineer, Bethlehem Steel Company, Bethlehem, Pa.

Figure 2 shows a comparison of the Tensile Requirements of the three comparable specifications. In general, the tensile strength ranges are closely overlapping and ductility requirements are almost identical. The proposed new specification has a maximum tensile strength which is 5,000 psi higher than A7 and A373 because of the higher yield point that must be realized. The 36,000 psi minimum yield point in A36 is the significant figure in this table.

The major point of interest to most of you is the comparison of Chemical Requirements shown in Figure 3. Specification A7 has only a maximum prescribed for phosphorus and sulphur, both residual elements. The identical maxima are a part of A36 and A373, but I have omitted them from this comparison so that the carbon, manganese and silicon requirements can be more easily followed. Specification A36 has a maximum carbon of 0.28 prescribed for all products in all thicknesses. This same figure is used in A373 for Bars and Shapes of all sizes, while the maximum carbon for plates ranges from 0.27 to 0.25, depending upon thickness.

This table enables us to see how closely A36 material approaches A373 material in chemistry and therefore weldability. In Bars and Shapes, the total effect of carbon and manganese is almost identical. In Plates the

SCOPE CLAUSES
Comparison of Three Structural Steel Specifications

A7	A36	A373
This specification covers carbon steel plates, shapes and bars of structural quality for use in the construction of bridges and buildings and for general structural purposes.	This specification covers carbon steel plates, shapes and bars of structural quality *not over 4" in thickness* for use in the construction of bridges and buildings and for general structural purposes.	This specification covers carbon steel plates, shapes and bars of structural quality *not over 4" in thickness* for use in the construction of *welded* bridges and buildings and for general *welded* structural purposes.

FIGURE 1

TENSILE REQUIREMENTS
Comparison of Three Structural Steel Specifications

	A-7	A36	A373
Tensile strength, psi:			
For shapes of all thicknesses	60,000 to 75,000	60,000 to 80,000	58,000 to 75,000
For Plates and bars up to 1½ in., incl. in thickness	60,000 to 72,000		
For Plates and bars over 1½ in., in thickness	60,000 to 75,000		
Yield Point, min, psi	33,000	36,000	32,000
Elongation in 8 in., min, per cent	21	20	21
Elongation in 2 in., min, per cent	24	23	24

FIGURE 2

CHEMICAL REQUIREMENTS

Comparison of Three Structural Steel Specifications

			Bars	*Shapes*	*Plates*
A7	C		...	...	...
	Mn		...	...	...
	P	(max)	.04	.04	.04
	S	(max)	.05	.05	.05
	Si		...	...	...

			Bars		*Shapes*	*Plates*		
			3/4" and Under	*Over 3/4" to 4"*		*3/4" and Under*	*Over 3/4" to 1 1/2"*	*Over 1 1/2" to 4"*
A36	C	(max)	.28	.28	.28	.28	.28	.28
	Mn		...	.60/.90	...	...	.80/1.10	.85/1.20
	Si		...	...	...	...	...	.15/.30

			Bars		*Shapes*		*Plates*			
			1" and Under	*Over 1" to 4"*	*Other than Heavy W. F.*	*Heavy W. F. 10" to 36"*	*1/2" and Under*	*Over 1/2" to 1"*	*Over 1" to 2"*	*Over 2" to 4"*
A373	C	(max)	.28	.28	.28	.28	.26	.25	.26	.27
	Mn		...	.50/.90	...	.50/.90	...	.50/.90	.50/.90	.50/.90
	Si		...	...	...	...	...	...	.15/.30	.15/.30

FIGURE 3

chemistry for the new specification is somewhat higher, particularly in the thicknesses over 3/4". However, both A36 and A373 offer a much greater guarantee of weldability than does A7. For practical purposes A36 is virtually synonomous with A373 for shapes and for plates up to 3/4" and therefore equivalent in weldability. It has been estimated by reliable sources that thicknesses up to 3/4" cover 90+% of all welded construction. It was this close similarity in chemical requirements that led to the decision by the Task Group to include a recommendation for welded construction such as is given in A373. This Sub-Committee II letter ballot indicated a strong majority in favor of such a scope clause. However, in view of five negative ballots, the reference to welded construction was deleted in order to expedite the adoption of the specification. After adoption, the possibility of inserting the reference to welded construction will be discussed.

In thicknesses over 3/4", the chemistry must of necessity be somewhat higher to produce higher yield strength. In these heavier thicknesses, the designer has a choice—closer control of welding technique of A36 or substitution of A373 for perhaps a very slight increase in weldability at a substantial decrease in strength. It should be pointed out that in these heavier thicknesses, geometry rather than chemistry is likely to be the critical factor in practical welding problems.

We should point out that the specification writers had a choice of uniform physical properties across the entire thickness range with chemistry graduated upward in the heavier thicknesses; or uniform chemistry across the entire thickness range, with physicals graduated downward in the heavier thicknesses.

It was determined that for purposes of design and estimating it would probably be better to have uniform physicals and, therefore, uniform design stresses for designing and estimating purposes. Here again, for welded structures the choice can be made in the heavier thicknesses between A36 and other specifications if the designer wishes to be ultra-conservative.

I have attempted to cover only the basic features of A36. Bend Test Requirements are the same as those of A7 and A373. Modifications for ductility requirements according to thickness are standard as are all dimensional tolerances.

Discussion

(*Q*) A. L. COLLIN, Kaiser Steel Corp.: Is there a comparison of costs between A7 and A36?

(*A*) LECRON: I would like to ask Mr. R. E. Wilmot, Manager of Sales (Structural Shapes), Bethlehem Steel Company, to answer this question, since it is really not in my field.

(*A*) WILMOT: I'm glad the question referred to "cost" rather than "price," because "cost" is the biggest part of your job. No cost information is yet available, but if A36 is certified by ASTM, and its values are to be incorporated into future designs, I can assure you that it will be economically better than our present A7 steel.

* * *

(*Q*) ANONYMOUS: Some architect-engineers do not consider A7 steel to be weldable. How can we combat this attitude?

(*A*) LECRON: I think that will have to remain a matter of opinion. With the A7 specification in its present form, there is no limit on carbon and manganese. It is impossible for me or anyone else to say that any steel mill in the country that makes A7 steel will always ship a product that will come under a certain limiting chemistry not called for in the specification.

In my own company (Bethlehem), I am familiar with the plates, shapes and bars rolled under A7 specifications, and I have every confidence that they are weldable under most conditions. However, the carbon-manganese content of A7 material supplied by each producing company may vary because of differences in metals, differences in residual alloys, and most of all, differences in finishing temperatures. These variations occur not only from company to company, but even from plant to plant within the same company. Therefore, I don't think we can say, unqualifiedly, that A7 material is always weldable.

The main reason for writing the new A36 specification with limited carbon content on all products and limited manganese content on heavier plates was to assure weldability of the new steel.

* * *

(*Q*) E. R. BABYLON, Kaiser Steel Corp.: The proposed specification contains a .28 maximum carbon. In your Bethlehem Plate Mills, what would the actual carbon range be for 3/4" and for 1 1/4" plate? Also, what are the future possibilities of controlled finishing temperatures on plate mills?

(*A*) LECRON: To answer your second question first, the controlling of finishing temperatures on modern plate mills is very difficult. The newer mills are designed for fast rolling and finishing temperatures above the ideal range. To slow down the process would result in severe losses in the tonnage rolled. The variations in width and thickness of plate make temperature control rather impractical. Your first question concerns ladle carbon ranges for 3/4" and 1 1/4" plates. Under average conditions, to meet the proposed specification, we would aim for .22 to .26 carbon for 3/4" plate and .20 to .25 carbon for 1 1/4" gauge (the 1 1/4" has higher manganese required than the 3/4" gauge).

THURSDAY AFTERNOON SESSION

May 5, 1960

Presiding: JOE BURKE, JR., Sales Manager, Sterling Steel & Supply Co., Denver, Colo.

Fabricating Machinery and Methods — A Symposium

FRED W. DULL
ROBERT C. KIDD
GEORGE W. HALL
RICHARD W. MOLTHOP
R. A. SHAW

Sawing and Drilling Equipment and Practices

FRED W. DULL

The subject assigned to me is "Sawing and Drilling Equipment", and it quite naturally follows that I would feature our new Boulton and Paul Semi-Automated Fabricating Line. However, before getting into the subject, I would like to digress for just a few moments and comment about some of our common problems.

We are all very well aware of the competitive situation that has existed for the past two years or more. It is certainly true that there has been, and will probably continue to be, an intense competition between each of us as fabricators.

The down-turn in industrial construction has had quite an effect on the available volume. In addition to this, we must be fully aware of the competition from pre-stressed and reinforced concrete, as these have been eating away at the volume which we used to enjoy. We have today still another competitive force looming ever larger on the horizon—imported fabricated structurals. With the St. Lawrence seaway now a reality, one can only ponder the extent of imports in the future. To further compound the problem, our industry, according to Iron Age Magazine, has, since 1957, increased fabricating capacity from approximately seven million to over eight million tons annually.

I am quite sure that every company represented here today has come to the same conclusion—that we must do something to upgrade the efficiency of our plants, to improve the output of our workers, as well as to reduce costs in many other areas of the business. Only by doing these things can any of us hope to compete in the future.

We at International Steel Co. recognized that many inefficiencies existed in our overall plant layout and flow of material—inefficient and, in some cases, insufficient equipment, as well as some outdated methods. We decided to do something about it.

Why did we make such a sizable investment in the Boulton & Paul Fabricating Line? Because it was obvious to us that something had to be done to improve our methods; to eliminate much of the scratch layout, followed by single punching or single spindle drilling. Although we had a modern and efficient spacing table punch, it was consumed most of the time on angle punching, as we do quite a volume of riveted truss work. We had a definite need for additional spacing table punching equipment for beams, etc., as well as some multiple spindle drilling equipment for heavy sections. The Boulton & Paul equipment came into the picture at just the time we were involved with this analysis.

To make a long story short, we made a careful layout of this equipment and found that it fit very nicely with some other plans that we had made to improve our layout and flow of material. We analyzed the savings anticipated and decided that it was the answer to our problems, as far as prefabrication of beams, columns, etc., was concerned.

Before installing our new system, two 18-inch Marvel hack saws, combined with a 24-inch friction saw, handled the bulk of our cutting. Most of our beam and column work requiring accurate length and square ends was cut on the Marvel saws. We handled all rough cutting on the friction saw. Following the cutting operations, nearly all of our layout was of the scratch method. Typical of many structural shops today, the layout man with the drawing went through the laborious job of locating and center-punching the hole locations. Following the lay-

Mr. Dull is Director of Manufacturing, International Steel Co., Evansville, Ind.

out, we would single punch or single drill most of the beams and columns handled in our shop. Needless to say, the cost of cutting, layout, punching and drilling, when combined with the many handlings and storages between these operations, offered quite an opportunity for improvement.

The Boulton & Paul equipment has considerably changed our old methods. Where we previously would send a full set of shop details to the prefabrication area in order that the layout men could properly mark the material, we now make up the web and flange drill dimensional layout sheets in the office. We retrained our staff of billers, one of whom you see in Figure 1 doing this work. The information is transferred directly from the drawing to the sheets, thereby eliminating the drawing in the shop. It was necessary that we slightly alter our method of detailing by adding some accumulative dimensions that we had not previously found necessary.

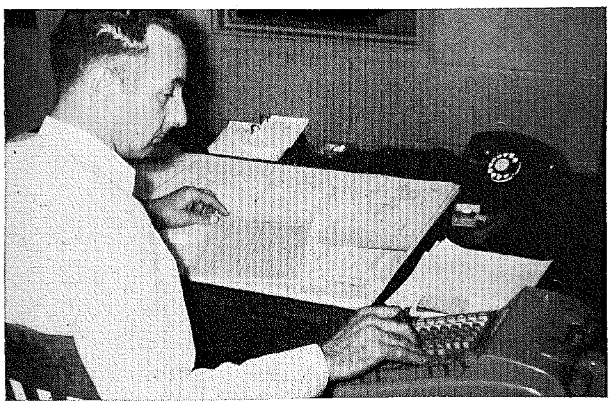

FIGURE 1

The biller in Figure 1 is using a calculator set up to handle feet, inches and sixteenths, thereby eliminating the need for a scratch pad and pencil.

Figure 2 shows a schematic layout of the equipment installation, which covers an area approximately 70 x 250 ft., in the extreme south end of our plant. The capacity of our installation is up to and including 30 inch wide flange 210 pounds and 14 inch wide flange columns 426 pounds, 50 ft. in length.

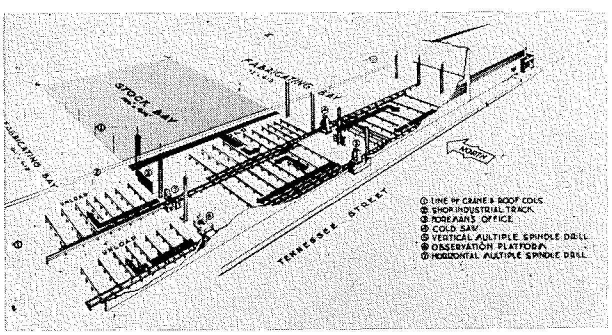

FIGURE 2

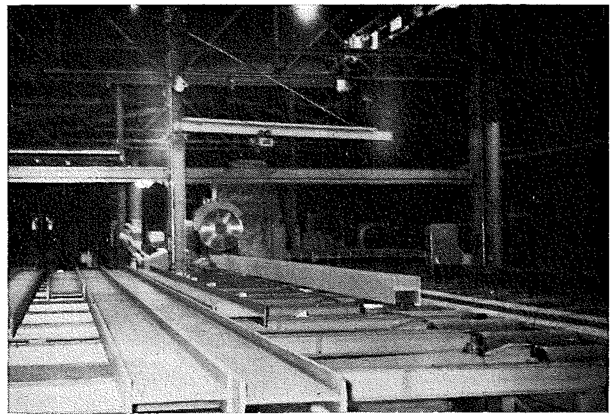

FIGURE 3

Figure 3 is a view taken from the extreme end of the feed conveyor, into the saw, which you see in the background. The column, when on the loading skids, is moved by a dog-type chain conveyor. The dogs are visible in this picture.

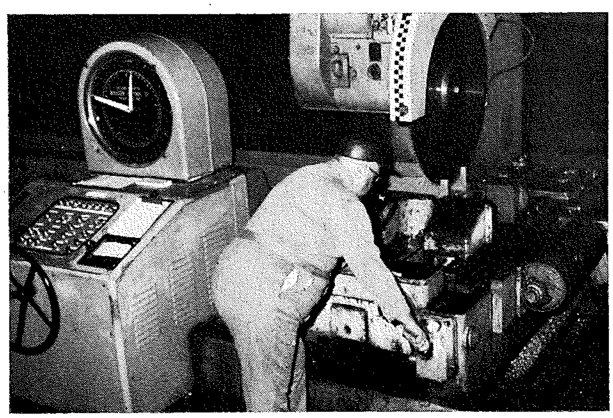

FIGURE 4

The saw operator in Figure 4 controls the movement of the steel from the bench onto the conveyor and through the saw for the length required. The measuring dial, shown in this view, allows the operator to set the correct length for the cut to be made. The material moves at 60 ft. per minute and, as the steel approaches the proper dimension, the operator can inch the material up to within a reasonable setting of the dimensions required. Exact settings are made by using the pilot wheel, seen in front of the console cabinet. We can cut lengths to plus or minus $\frac{1}{32}$-inch without difficulty. When the operator has the material at the proper setting, the self-centering vises are closed by a button on the console and he moves approximately two steps to operate the hydraulic saw control.

Figure 5 shows the proper relationship of the saw, which is at the right, and the vertical drill, which is at the left. The small I-beam to the extreme left is on the

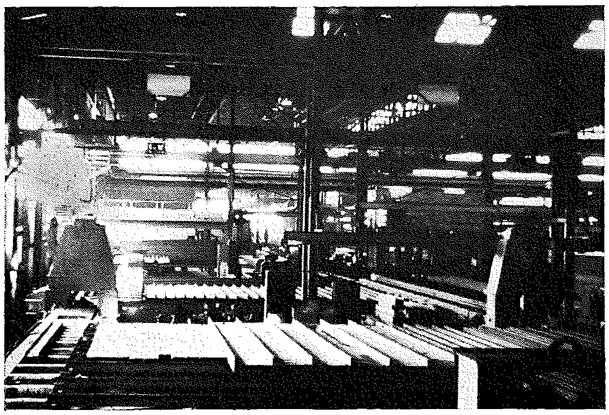

FIGURE 5

feed conveyor to the vertical drill. After the saw operator has unloaded the piece cut, depositing the steel on the skid, further movement is controlled by the vertical drill operator.

The operator of the vertical drill sets the longitudinal spacing in the same manner as the saw operator. In Figure 6 you can see the sheet on the clip board which gives the operator the settings for drilling the material.

Figure 7 shows the six spindles, two of which are

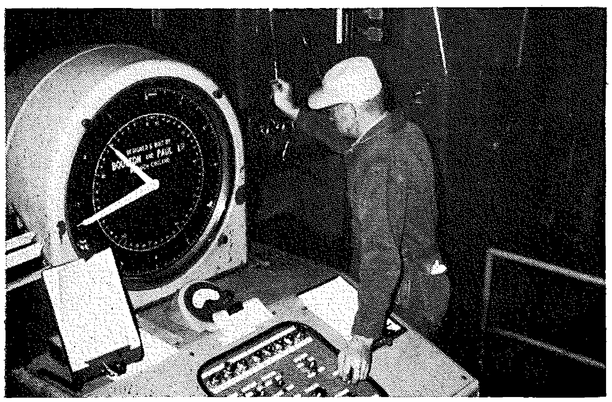

FIGURE 6

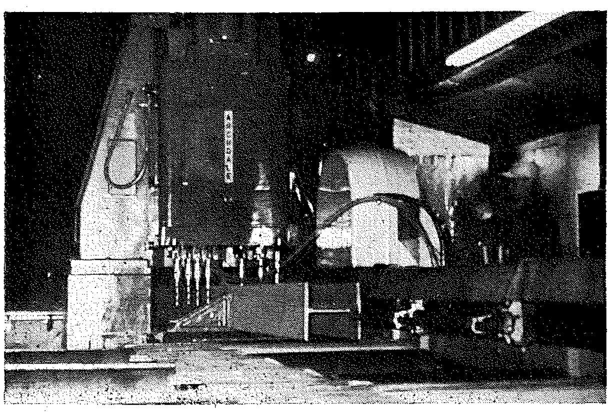

FIGURE 7

being used on this particular column. The proper gage is set from the fixed side of the vise to the right. The vise, visible to the left, which is hydraulically operated, opens and closes to any size within the range of the machine. The cable that you see extending from each spindle is a gagging device which allows the operator to select the required spindles from his operating station.

The operator sets the required gage by adjusting the spindles as indicated on a clock dial for each spindle. The spindles in the vertical drill are adjustable down to a $2\frac{1}{2}$ inch minimum, and will handle $\frac{5}{16}$ through $1\frac{1}{4}$ inch diameter inclusive. The six spindles are lettered A, B, C, etc., and the dials are lettered the same.

The column is unloaded by the operator from the web drill conveyor, and he advances it, by selecting the proper button on his console, to a conveyor which unloads the material after being web drilled only. The unloading operation is completely automatic, once the web drill operator pushes the proper button.

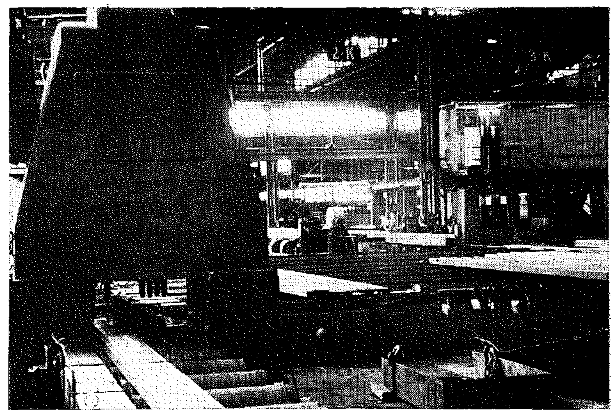

FIGURE 8

Figure 8 shows the relationship of the web drill and the flange drill with the transfer bench in between. The movement of the material once unloaded from the web drill, is under the control of the flange drill operator, and he advances the material onto the feed conveyor as required. You will note in this view a channel entering the web drill. We run channels with the toe down in order to convey them with the pusher dogs. The movement of the steel is handled in a similar way as on the saw and vertical drill. Two operators are required on the flange drill. One man handles the movement of the material as well as the longitudinal settings required, and the other operator makes only spindle adjustments. Each operator has duplicate copies of the layout sheet so that they work in unison.

Figure 9 shows the two six-spindle heads of the flange drill, made up of an upper and lower bank of three spindles each. The spindles are adjustable horizontally or lengthwise, from $2\frac{1}{4}$ inches minimum to 6 inches maximum. They are adjustable vertically, which estab-

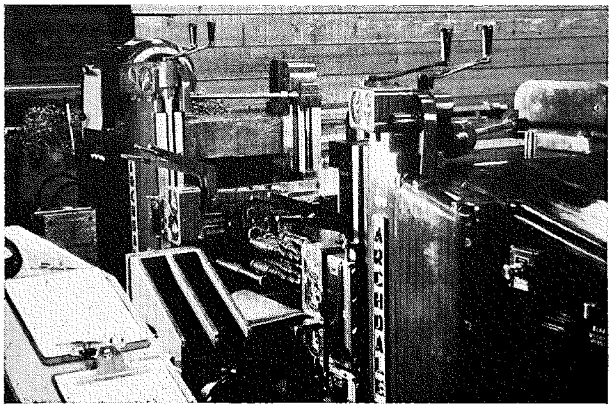

FIGURE 9

lishes a gage on the flange, from $2\frac{1}{4}$ inches minimum to $14\frac{1}{4}$ inches maximum. The spindle spacings longitudinally are set according to the lower clock dials, one dial for each spindle. The vertical adjustments of each bank of three spindles, in order to establish the proper gage about the center of the web, is made by using the two clock dials at the top of the machine. The operator can select any number of the six spindles on either side

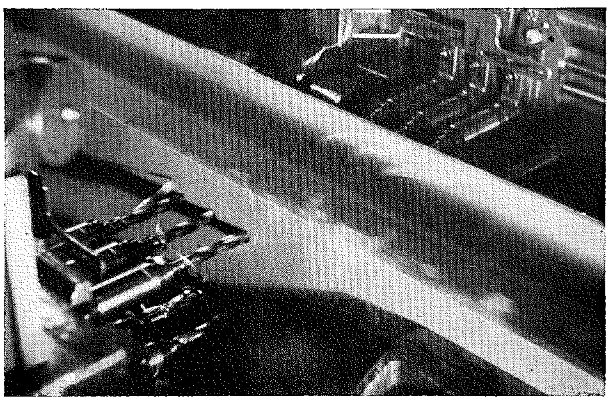

FIGURE 10

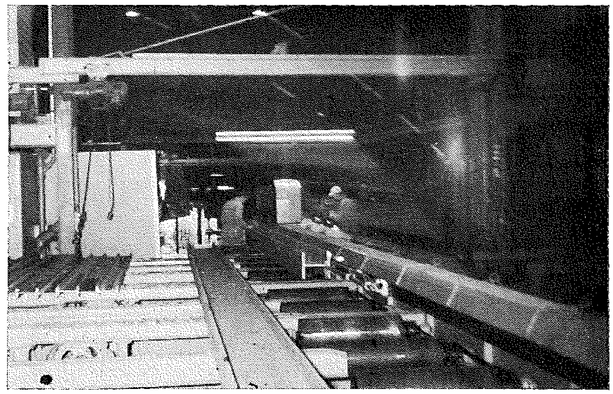

FIGURE 11

of the machine as required. Figure 10 shows all 12 spindles entering the column flange.

Figure 11 shows the column coming through the flange drill in the unload side of the conveyor, and bearing against the measuring device which sets the longitudinal dimensions on the clock dial. When drilling is completed on the flange drill, the operator needs only to select the proper button on the console to automatically unload the material onto the skid on the left.

It is probably quite evident that this was a rather costly installation, and it follows that there must have been a sizable savings from the investment. Now, what have we accomplished?

First, in our prefabricating operation on all beams, columns, channels, etc., within the range of this equipment, we have eliminated all drawings and templates. Second, we have eliminated more than half of the previous operations and handlings, and, as you can appreciate, they were the more expensive half. Third, we have eliminated the layout man and, to sum it all up, we have reduced the previous cost of prefabricating the items within the range of this equipment by approximately 75%.

Forming Equipment and Practices

GEORGE W. HALL

During these times of high labor cost, every progressive fabricator is looking for better equipment and methods. In order to remain competitive, it is not only desirable but it is also a necessity to have up-to-date pressing and forming equipment. Today I will merely touch on some of the more important pieces of equipment which fall in this category. In particular, I would like to discuss plate rolls, angle rolls, bulldozers, press brakes, and presses.

Plate Rolls

The general appearance of plate rolls has changed very slightly during the past 50 years. However, many new features have been added to improve the productivity of this type of equipment.

Figure 1 shows the two basic types of plate rolls—pyramid and pinch.

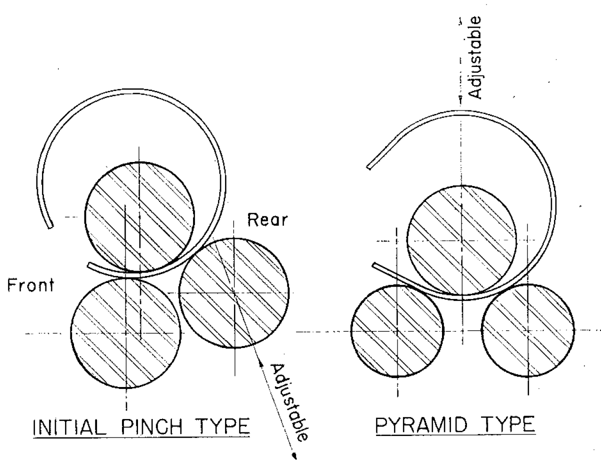

FIGURE 1

First, let us talk about the pyramid type roll. The two lower rolls are in a fixed position and both are power driven. They are smaller in diameter than is the top roll. The top roll is adjustable up and down to determine the diameter of cylinder produced. The pyramid roll has only two main advantages over the pinch type; namely, it can be provided with dies and used as a press brake, and it is cheaper than the initial pinch type.

The initial pinch type roll has three forgings of the same diameter. The two front rolls are known as the pinching rolls. The top roll is in a fixed position. The lower front roll is adjustable up and down to suit the thickness of the plate being rolled. The rear roll or bending roll is adjustable up and down to determine the diameter of the cylinder produced. All three rolls should be power driven.

Over 90% of the rolls purchased today are of the initial pinch type. Many of these are ordered to replace pyramid type rolls.

The pinch type roll will form more nearly perfect round cylinders than a pyramid because of the fact that the flat spots on the ends are practically eliminated. It will also form smaller diameter shells over given roll forgings than the pyramid. The pinch type is faster.

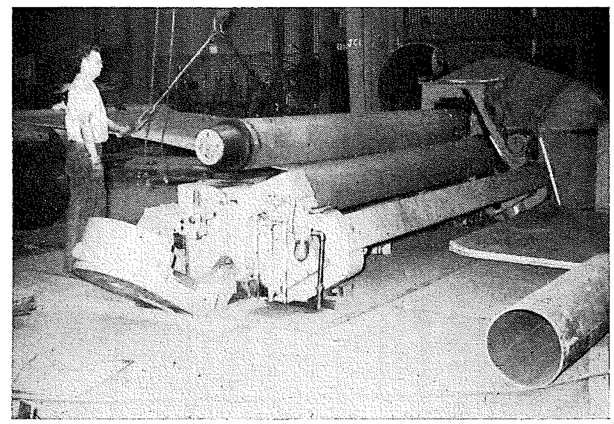

FIGURE 2

Pre-bending of most diameters can be done on the pinch type and it is necessary to do this pre-forming on an auxiliary piece of equipment when using the pyramid roll. Figure 2 shows a very new pinch type roll. Cylinders are being rolled in this particular application. As a short cut to subsequent operations, this tube is fitted and tacked in the roll while the plate is still under pressure. This picture also shows the drop end hinge which has been lowered to facilitate removal of the rolled pipe. The drop end hinge is pneumatically operated. The pinch type is opened in a few seconds and this operation automatically tilts the top roll and releases the pinch over the full length of the plate. The pyramid is opened by adjusting the top roll upward, and then lowering the hinge; this requires several minutes. Similarly, the closing of the pinch roll is much faster, and it is automatically returned to its original position.

The pyramid type machine, with an idler top roll, is driven by friction of the plate. Therefore, depending upon the size of the machine, you may have trouble in trying to roll material as thin as $\frac{3}{16}$ and it will usually not handle $\frac{1}{8}$-inch material. The bending pressure from the light plate is not sufficient to resolve the idler top roll. In like manner, the pyramid will mar and scratch any thickness of soft or polished material. The pinch

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roll will handle thin material and it will not scratch soft material because of the positive pinch and because all 3 rolls are power driven.

Plate rolls can be grooved for rolling angles, tees, and bars. This type of machine application can save the purchase price of an angle roll for the fabricator who works with light materials.

The new dial indicator gages are a most helpful piece of equipment in the use of a pinch roll. These indicators permit predetermined settings for rolling duplicate pieces and it is possible to save considerable time and to eliminate many passes of material through the roll by the use of this gadget.

Roll out tables and other handling equipment are a very necessary consideration in the installation of a roll set-up. A gantry crane is more satisfactory for a rolling operation than is a jib crane.

The actual machine rolling time is about 50% of an entire rolling cycle. Therefore, it is vital to have a fast machine, and we believe that, for most applications, a rolling speed of 30 ft./minute is practical.

Angle Roll

An angle roll is a must for a medium size and/or large fabricating plant. It provides the fastest known method for rolling small bars, channels, angles, tees and beams.

An angle roll is a very expensive machine in larger capacities, and it would not necessarily be the best buy for a small shop that wanted to do rolling. Further, large machines require deep heavy foundations and, in such installations, the machine is no longer portable.

Angle rolls come in both vertical and horizontal models. The usual preference is for a machine with vertical roll shafts wherein the material is handled in a horizontal plane. Figure 3 illustrates such a machine.

Unless material is pre-formed prior to rolling, usually there is a waste on each end of the material being rolled because of the flat spots. These flat spots must be cut off and scrapped.

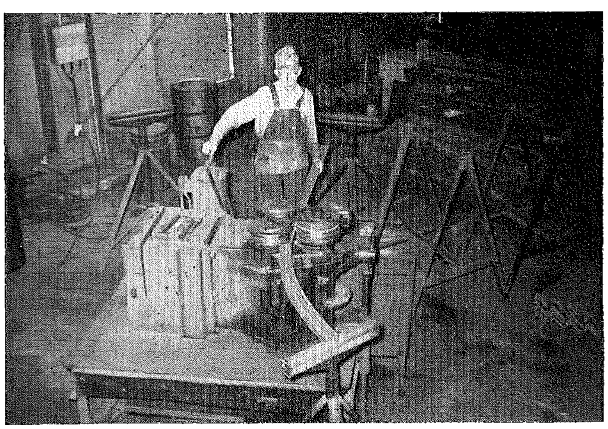

FIGURE 3

I do not know of any revolutionary developments in the angle rolling field other than new types of controls, new types of steel for the rolls, quick acting clamps for roll removal and replacement, and portable rolls for handling material in and out of the machines.

If a plant is operating on a limited budget for capital expenditures, it may be wise to pass up the angle roll and buy an all purpose machine such as the bulldozer for rolling small shapes.

Bulldozers

Next let us talk about bulldozers.

Mechanical bulldozers are almost a thing of the past. Although the mechanical bulldozer is a useful tool, it is a costly machine to operate because of control problems, slow adjustment of the stroke and other reasons. It is also very easy to overload this type of machine.

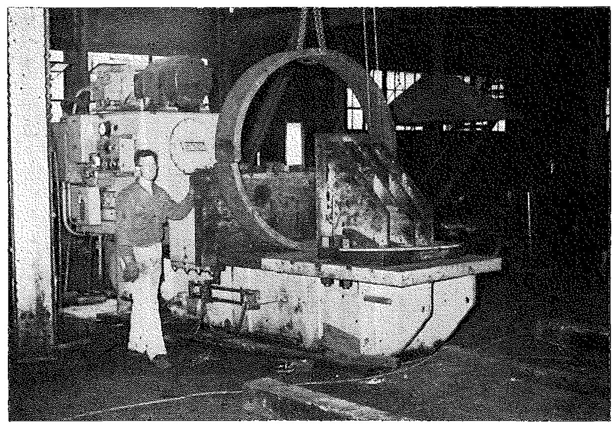

FIGURE 4

Modern plants are switching to hydraulic bulldozers. Figure 4 is a view of a Verson 200 ton 'dozer.' This machine is portable and it has an adjustable backstop. Unless the control end can be located in a leanto, it is advisable to mount the hydraulic mechanism, pumps, etc. at floor level in order that they will not be hit by cranes moving loads overhead.

Modern day features should be incorporated in new bulldozers. These include abrasive wearing strips, adjustable backstop, limit switches for determining length of stroke, etc. A bulldozer can also be partially automated. It can be operated manually by conventional methods. Further, it can be set and operated on a semiautomatic basis which means that by pressing one button, the head stop or ram will move forward to a predetermined position, build up pressure, return to initial position and stop. For repetitive work, it can be set and operated in the fully automatic fashion. This means that by pressing one button, this ram will advance forward, build up pressure, return to starting position, pause and then repeat the cycle. It is amazing how much this feature will increase productivity.

Handling is also a problem with a bulldozer. Portable roller stands and steel tables on the input and output sides of the machines can be used in conjunction with conventional hoisting equipment.

The bulldozer is one of the most versatile of all plant machines. It can be used for straightening, punching, cambering, rolling, bending, etc. For the money invested, a bulldozer is one of the best pieces of plant equipment that can be purchased.

Press Brakes

The trend today is definitely towards hydraulic press brakes. This is particularly true when fabricating material ⅜-inch thick and heavier. Some of the advantages of a hydraulic press brake are as follows:

1. Less skill is required to operate it.
2. It cannot be overloaded.
3. It is about 10 to 15% cheaper than a mechanical brake.
4. It can be designed with a longer stroke.
5. It is easy to regulate the pressure.
6. The stroke adjustment is much faster than on a mechanical brake.

(*At this point speaker showed slides covering use of plate brakes for rolling plates, punching, forming, etc.*)

In buying any type of press brake, it is important to provide for an extension of the ram and bed such as shown on the ends of this brake (Fig. 5). These outriggers are very important for bending closed units; there is no stripping problem.

Handling represents over half the cost in a press brake operation. It is possible and practical to reduce handling and perform several operations with one master set-up on a press brake. Double decker dies can be used for this purpose.

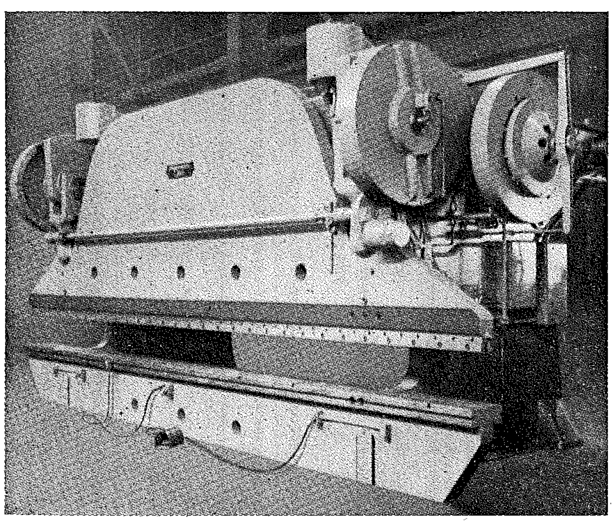

FIGURE 5

Good dies, imagination and effective use of gages and backstops are very necessary in obtaining maximum production on press brakes The average foreman can design and provide such gages without too much supervision.

As I have stated, the hydraulic brake is 10 to 15% cheaper than a mechanical unit. It is a very versatile machine and can be used for rolling, coning, bending, and punching. Today's modern machine has the hydraulic manifold block mounted at the rear of the machine along with pump, motor and reservoir; leaking oil lines are no longer a major problem.

Hydraulic Presses

The only types of presses which we will discuss will be those that are of hydraulic design. These presses can be either of the four post variety or of the "C" frame variety.

Figure 6 shows an old four post hydraulic press. The normal method of handling work through the press is from front to back. You will note that the material can also be handled from left to right. This is an advantage when straightening or cambering long material.

(*At this point, speaker showed slides covering the use of hydraulic presses for rolling plates and angles, straightening structural materials, etc.*)

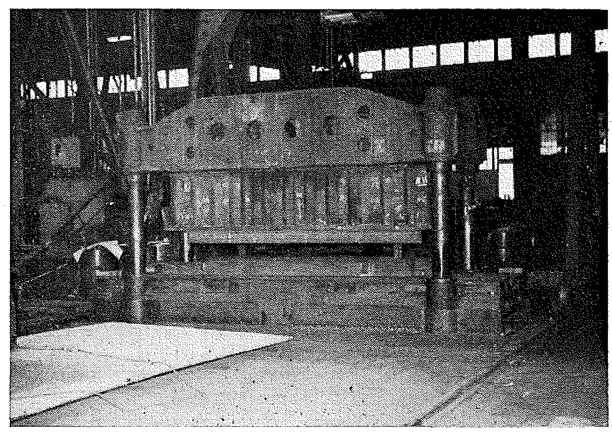

FIGURE 6

The modern "C" frame type press can be equipped for manual, semi-automatic and fully automatic control. I have described this type of automation in the discussion on bulldozers.

The decision on whether to buy a "C" frame or a four post press depends strictly upon the end use of the equipment. The open throat or "C" frame press is chosen for many applications for the following reasons:

1. It is a very simple operation to change the dies in this press.
2. The open throat permits an unlimited range of width and length of material being processed.

3. It is simple to provide this machine with a horizontal ram for flanging.
4. It is also a very good machine for straightening and the normal run of pressing.

This type of press will open up under capacity load conditions, and the resulting deflection creates some problems when rolling heavy shell plates.

The four post press is the best buy on today's market. In some cases, depending upon throat opening, the four post press will only cost ½ to ⅔ as much as the "C" frame model. The operating speed of the four post machine is as fast as the "C" frame unit. There is no problem with eccentric loading of dies and no worry about scoring of the hydraulic cylinders when this machine is used.

In a complete pressing cycle, handling can amount to as much as 60% of the total operation. The actual pressing time is about 25% of the total. It is therefore important that both handling equipment and speed of pressing be the best obtainable. We have found that gantry cranes can do the best job in handling material at a press.

Conclusions

Here are some typical structures which have been fabricated by means of the aforementioned equipment. (*Speaker showed slides of finished products made up of cone plates, shell plates, rim angles, structural columns, etc.*)

I have attempted to cover the basic machines that are used in a normal fabricating plant for pressing and forming. I hope that I have covered items that have been of interest and value to you. Thank you.

Numerical Control for Structural Fabrication

ROBERT C. KIDD

The words "Numerical Control" have come to be synonymous with progress and cost savings. Certainly, Numerical Control has created more interest and has captured the imagination of manufacturing management as nothing else has in the past few years. It has been estimated that the machine tool industry will sell from 15 to 25 million dollars worth of numerically controlled machine tools in 1960 and about 50 million in 1961 and 75 to 100 million in 1962.

What is this phenomenon called "Numerical Control" and what will it mean to the structural fabrication industry? In this paper, which is much too short to possibly cover such an important and vast subject, I will attempt to show what our company and a segment of our industry have done, and what I feel our entire industry will be doing in the next few years as concerns automation and Numerical Control.

Let us first define Numerical Control. Numerical Control is a form of tooling that controls the motions of a machine by numerical values stored in a suitable medium. Numerical values normally are in a binary or two number system and the suitable medium is either a punched card or punched tape, although magnetic tape is sometimes employed.

There are two distinct types of Numerical Control:

1. Point to point, or discrete positioning
2. Contouring or continuous path

We will be dealing with point to point positioning as it will apply to structural fabricating equipment encompassing those processes with which we are most familiar, such as punching, drilling, sawing, etc., for in these processes we are only attempting to work in a single plane or two dimensional system rather than a three dimensional system.

Why should we be concerned with Numerical Control for structural fabricating equipment? I feel the answer lies primarily in the great demand to reduce fabricating costs which is sweeping our economy. Our industry has lain dormant for many years with regard to the modernization of equipment and automation and it has only been within the past few years that any effort has been made to "change from the old tried and tested way" of doing things in the shop. You can go into plant after plant in our industry and find them doing things the same way they have done them for decades. This all too apparent attitude is changing rapidly, however, and Numerical Control is going to accelerate the change. Better and more efficient methods of handling steel are bringing us to the threshold of automation. The Boulton and Paul automated fabricating plants now in operation at the International Steel Company and the Mississippi Valley Structural Steel Company are indicative of this trend. Now, incorporate Numerical Control, which automatically feeds in the proper information to a machine, telling it what to do so that all or most of all human "error" has been removed, and we have the ultimate as we can foresee it today.

We are going to discuss types of punching and drilling equipment that have been developed for our industry so that you can see just what has been accomplished to date. This does not mean that the field has been narrowed to these two operations nor does it mean that the specific applications you will see constitute the only efforts made. On the contrary, we feel that all areas of shop fabrication will some day employ Numerical Control directly or they will be in some way indirectly controlled by automated data processing equipment. This means shearing, cut-off, welding, assembly, cleaning and painting can be areas where automation, using Numerical Control, can be considered. In fact, we will attempt to show how this whole subject can reach to the engineering department and the drafting room.

The two most logical areas to begin this process of integrating Numerical Control into fabricating equipment were in punching and drilling, where a definite point to point relationship exists and where potential labor savings were high. The old methods of layout and simple punching or drilling normally required two men to lay out the hole spacings, and two men to simple punch, or one man to drill, depending on the thickness of the material. Punching has been accomplished on all types of rack punches, punching one hole at a time, gate punches, where multiple punch set-ups can be made, or large open throat or guillotine type punches with spacing tables to move the mass through a multiple punch system. Practically all of these methods use two, three, and sometimes four men to operate and handle material. All of them employ a method of setting and reading a template or reading a drawing directly. Regardless of the method, they all require considerable time and substantial labor dollars to prepare the information for actual operation of the equipment and the element of human error has been compounded in most cases many times.

To begin this process toward automation, Mississippi Valley Structural Steel developed a one man operated angle punch (Figure 1) which is electro-hydraulically actuated and performs in the following manner:

The operator sets a plug for each hole to be punched in a perforated plate attached to a moving table which is powered through an electro-hydraulic system. He clamps

Mr. Kidd is Industrial Engineer, Mississippi Valley Structural Steel Co., Melrose Park, Ill.

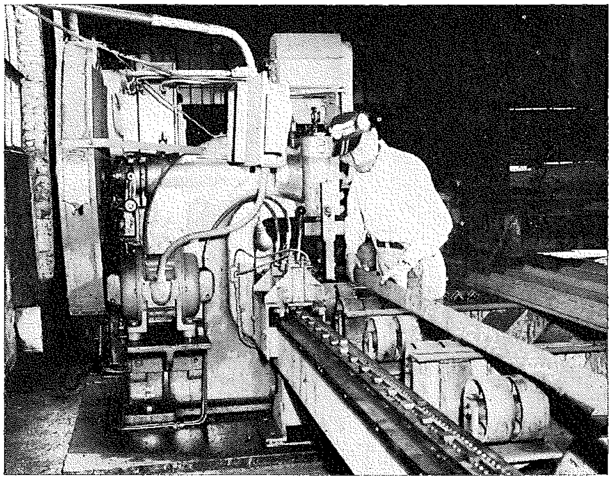

FIGURE 1

the angle in position (Figure 2) and pushes a start button after he has pre-selected the ejection distance on the discharge side of the machine. The proper ejection distance is determined by the length of the angle being punched. The angle moves through the machine (Figure 3) accelerating to a maximum of 120 feet per minute between holes and decelerating to 12 feet per minute as it approaches the hole to be punched. The angle passes under a control station which automatically selects the hole, accelerates and decelerates the carriage, and triggers the punching cycle. After the punching operation is completed, the angle is automatically ejected from the machine at the pre-selected distance and the carriage returns to the start position. The advantages gained are primarily that one man can operate the machine and during the punching cycle the operator is released to perform other work, such as placing the next angle in position, marking copes, or putting the paint spots and piece marks on the last completed piece. All functions of the ma-

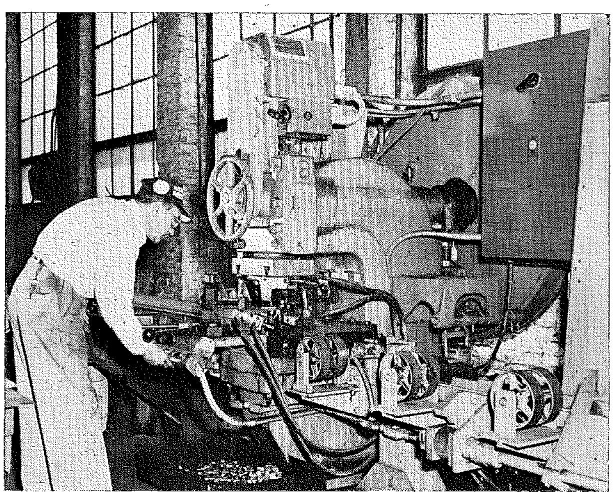

FIGURE 2

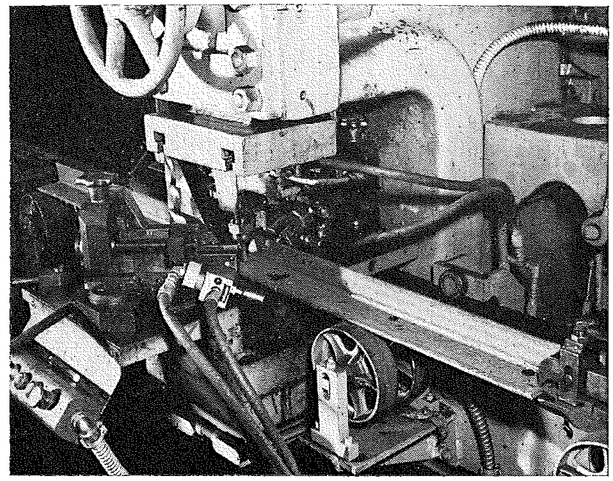

FIGURE 3

chine speeds, timing, etc., are easily adjustable and very accessible to the operator.

While we reduced this particular operation to one man and improved handling methods, it still was not the ultimate in automation that we desired. Our next approach was in the area of beam punching, for we felt that if we could control the movement of large masses at fast speeds, completely automatically using Numerical Control, we could then adapt this basic principle of design to all phases of punching and drilling.

The first major obstacle to overcome was the proper selection of the medium we would use to supply input information to the actuating equipment. That is, from our definition of Numerical Control, our question was: *"What would we use for the numerical values stored on a suitable medium for this operation?"* We resolved the question down to the two most popular methods, tape control or card control. We selected the card control for the following reasons: First, it was easy to handle for both the preparation of the card itself and more important its ultimate use on the machine. Material flow in a structural shop, as you all know, is not uniform. In general, we do not produce a "standard product," and sizes, types, and quantities vary considerably. In tape control, the information placed upon the tape is continuous. To be most efficient the pieces coming to the machine would have to be in the same order as the information placed upon the tape. This limited us too drastically as to the ease with which an operator could "select" his work sequence. The right piece would have to be at the machine at the right time. With a card, it does not make any difference which piece comes first, since it is only a matter of selecting the proper card. We were able to obtain the most efficient piece of card preparation equipment and most efficient card from the Industrial Timer Corporation of Newark, New Jersey. The card we used is shown in Figure 4. We obtained twice as many discrete positions with this card (60 to be exact) than was possible from

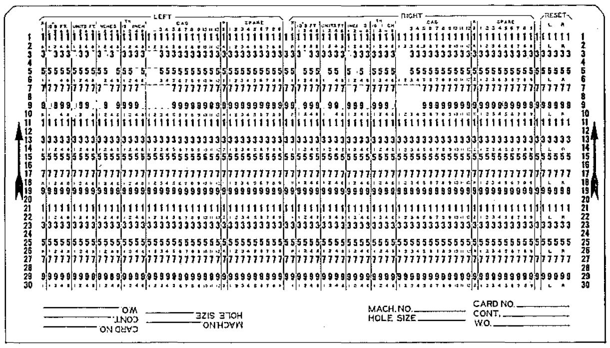

FIGURE 4

other sources. We superimposed on the basic card our own terminology and design dictated by the functions desired. We had determined that in over 90% of the cases, one card would be sufficient to provide information for the punching of both flanges and the web of a beam or channel. In the balance of cases, or 10%, two cards would usually do the job.

The Kling Brothers Engineering Company were approached to provide a suitable guillotine punch which could handle a wide range of beams and channels efficiently. As a result, the punch shown in Figure 5 was developed. This punch can handle from 6 WF 25 to 30 WF 210 pound beams and has several important features which make it compatible with Numerical Control. These include an air-brake clutch system and a balanced main drive system. The spacing table itself was devel-

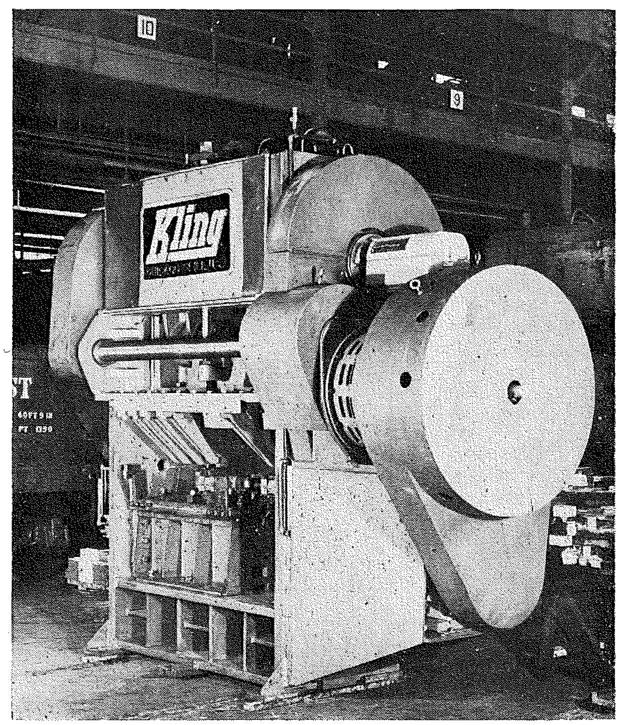

FIGURE 5

oped by Mississsippi Valley Structural Steel, and is completely electro-hydraulic. The Numerical Control system was developed in conjunction with Sanders Associates of Nashua, New Hampshire.

The process begins by preparing the card with the card punch shown in Figure 6. The operator places a card into the punch, reads the hole spacings directly from a drawing and punches the proper dimensions by depressing the keys on the keyboard (Figure 7). A luminous read-out device shows the dimension selected and serves as a visual check. The punching portion of the preparation equipment shifts the card from the left to the right hand side since the card is divided into two sections. The feet, inches, and fractions of an inch, plus the number of holes to be punched are selected. The gag section automatically selects the proper punches on the machine. Twelve punches can be so selected automatically with a possible expansion to twenty-one if so desired. The card punch automatically converts the fractional information so selected on the keyboard into a binary system which is used on the card. The card is properly identified and is ready for the shop.

At the machine the piece to be punched is placed on the feed-in side of the hydraulic lift table of the machine. The operator places a card in the card reader and sets the reader by depressing the start button. The carriage, which contains the hydraulic clamping equipment, clamps the

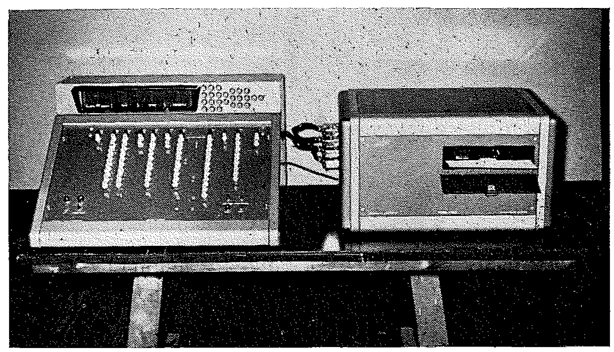

FIGURE 6

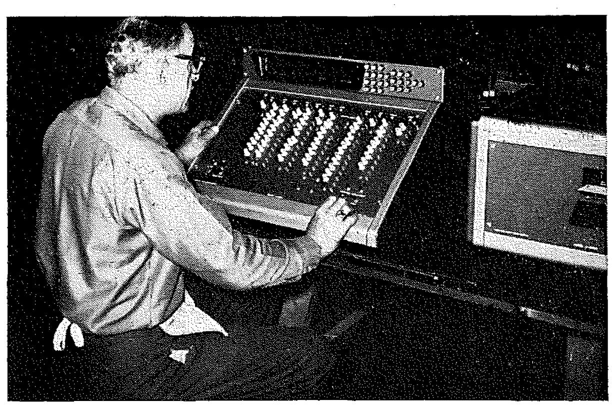

FIGURE 7

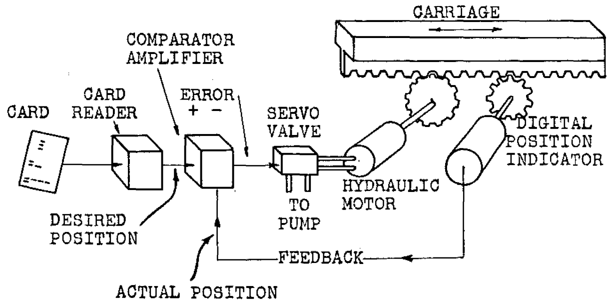

FIGURE 8

piece and is hydraulically driven through the machine. The operator pushes the start button and the piece moves through the machine at about 150 feet per minute maximum speed until it reaches the desired position. The card signals the proper selection of punches through the hydraulic gag system and the punch is automatically engaged. The holes are punched and upon the return of the punch ram to the top of the stroke the carriage moves to the next position. Selection of the proper position is accomplished in the following manner:

Figure 8 schematically illustrates the positioning operation. The card indicates an error signal which is electrically transmitted in milli-amperes of current from the card reader to an amplifier and comparator. The signal is amplified and sent to an electro-hydraulic servo valve attached to a fluid motor. This drives the main drive pinion through a reducer until the *0 error* point or "null" is reached. As this "null" point is approached, the error is constantly diminishing and the actual position or location signal is being transmitted back to the amplifier through a feedback system also attached to the drive rack. This accumulative signal is transmitted back to the comparator by means of this ratio tran. The signals are compared and less and less oil is allowed to pass to the fluid motor by the electrical signal input or control to the servo valve. Finally, the zero error or "null" point is reached, and the carriage is stopped and the punching cycle begins. This process is repeated until the system is returned to the "start" position. It takes approximately five seconds to select and punch any given hole selection. After a face has been punched, the carriage automatically returns to the "start" position and the piece is rotated and re-engaged to punch the next face. After all faces have been punched, the piece is removed from the machine. The operator is freed from the machine during the punching cycle to perform other operations. The total overall time to punch a given piece should be reduced by at least 50%.

Accuracy is an extremely important factor. Tolerances between hole spacing can be kept to a minimum of ($\pm$) .0005. This is obtained through this special drive and feedback system working in conjunction with the electronic control and electro-hydraulic servo system.

Eventually, when conveying equipment is made available to this operation, the beams will be transported to the numerical control punch on a conveying system coming directly from the cut-off operation.

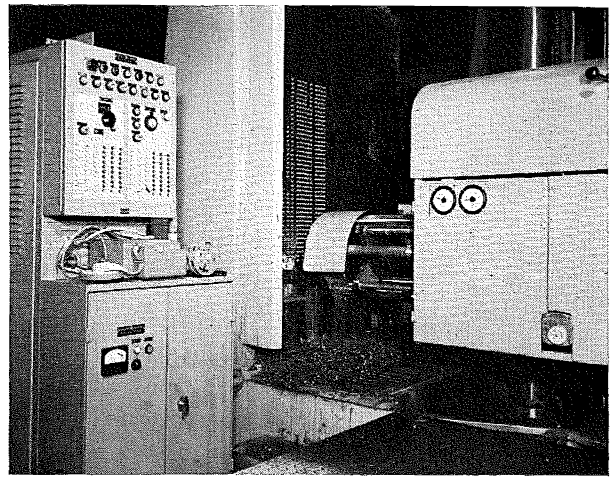

FIGURE 9

The same advances can be made and have been made in the drilling operation. The Walter P. Hill Company of Detroit, Michigan, has developed a tape controlled high speed drill which can employ the same basic principles as we have outlined in punching. The input medium is by means of a punched tape (Fig. 9). The Hill Company has incorporated a General Electric Mark IV (a four coordinate) system with an excellent electro-hydraulic circuit. The drilling capacities of the equipment are far beyond those normally experienced in the structural fabrication industry. Such feed rates as 30 inches per minute in mild steel are being accomplished without difficulty. By incorporating the proper conveying equipment to and from a machine based on this principle, entire girder sections can be drilled from a solid, all by one man, who merely inserts the 8 channel Friden punched tape in the tape reader and watches the operation as three drills, one vertical and two horizontal, drill simultaneously. With this high degree of accuracy it will be unnecessary to ream any sections so drilled. The basic drill design is flexible enough to accommodate built-up sections, large wide flange beams, and girders all on the same conveying system. Smaller detail sections such as base plates, gusset plates, etc., can be stacked drilled in reasonably large quantities at the same high feed rates and with tolerances of ($\pm$) .0005 accuracy. One large fabricator has already contracted to install this type of equipment and certainly more will follow suit as the apparent cost savings are realized.

Finally it can be seen, I feel certain, that a basic pattern is developing in our structural plants whereby Numerical Control can offer a means to control machine operations and with proper conveying equipment, steel can be moved from cut-off to final assembly, all automatically. At the final stage of assembly, we feel certain that improved as-

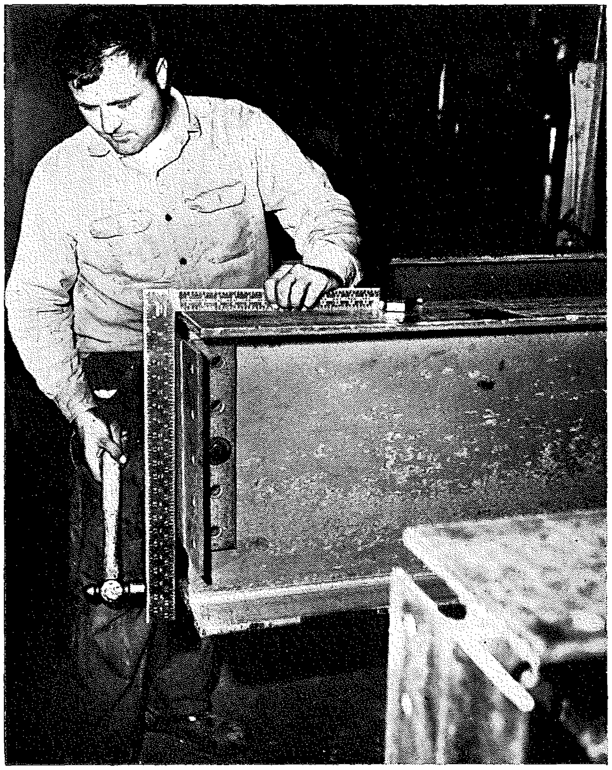

FIGURE 10

sembly operations can further provide savings such as the application of high strength bolts in lieu of riveting and the pieces moved to and through a cleaning and painting operation without ever touching the ground. In Figure 10 the fitter is assembling the end connection angles to a beam. He then aligns the connection properly after inserting one bolt. The entire connection is bolted and tightened with a torque controlled wrench. The completed product can be moved directly to the shipping yard without rehandling to a rivet operation. This entire process has become even more attractive since the acceptance by the AISC of the new high tensile bolt specification described this morning by Mr. Estes.

It is absolutely impossible to cover completely this broad field of Numerical Control and automation. However, let me close by saying that recent developments show that the future holds some very bright prospects for reduced costs in data processing and engineering design by employing computers in the engineering department to feed in basic design information and obtain cost estimates, plus all of the data necessary to provide the shop with machine tool control and operating information complete to the shipping lists for a given job. Figures 11 and 12 illustrate what one company is doing to approach this goal. It is not impossible. Boulton & Paul Limited are developing a system of processing information from general arrangement drawings directly to drilling plant information sheets with the assistance of a computor. While development is still in the early stages, sections have already been proved and it is anticipated that in eighteen months or two years the full program will be offered to owners or purchasers of the Boulton & Paul mechanized fabricating plant. Already much is being done to provide design programs for computers as per the Technical Bulletin No. 244 of 1959 issued by the American Road Builders Association, entitled "Standardization of Highway Bridges." Here in the computer program library are available a listing of highway bridge design programs for various types of computers. All of this is coming as certain as man will some day master the distance of 240,000 miles between this planet and the moon, and the structural fabricating plant of the future will have a new face and most likely that new face will be an extremely automated one at its best.

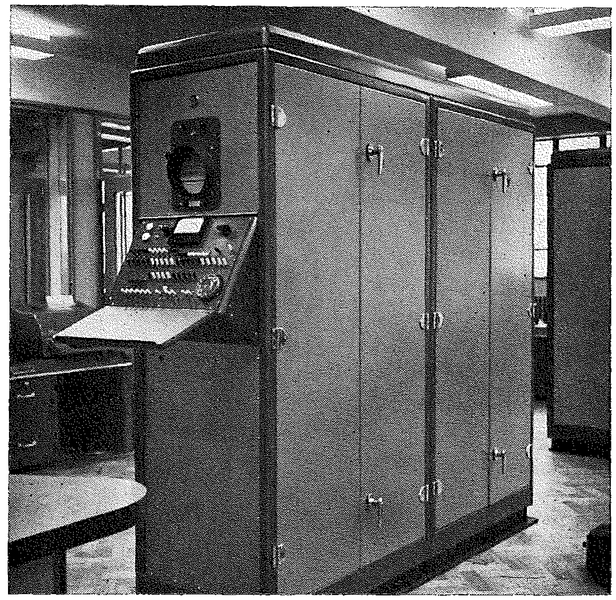

FIGURE 11

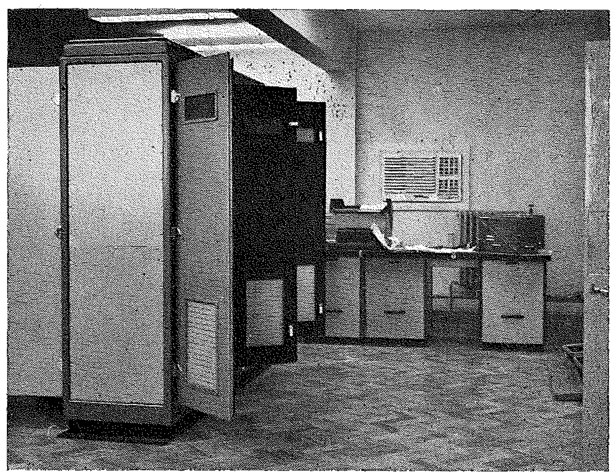

FIGURE 12

Carbon Dioxide Gas Metal Arc-Welding

RICHARD W. MOLTHOP

Gas metal arc welding is a family of similar welding processes which has come to be used more and more in the past few years. Recent work by Becker and Christopher* of International Harvester Manufacturing Research will go a long way towards pointing out how we can reduce welding costs by large factors. Mr. Becker kindly let me use a great deal of the material that he has prepared.

The family of gas metal arc processes uses three gases —argon, helium and carbon dioxide. The availability and cost of helium removes it from consideration in carbon steel work and the cost and lack of inherent penetration eliminate argon. Carbon dioxide is, therefore, of primary interest. CO_2 is cheap and the arc is very penetrating. The CO_2 processes are the bare wire CO_2, the flux cored CO_2 and the magnetic fluxed CO_2. The penetration that is inherent in the process is due to the CO_2 gas and the large current densities are approximately the same with all three of these processes. What I want to show is that (1) you can weld with CO_2 processes with no greater cost and in most cases with a savings of as much as 20% using standard welding procedure, and (2) that taking advantage of the inherent penetration of the welding processes by reducing fillet sizes quite conservatively and using butt and groove details tailored to the process, we can radically reduce welding cost, distortion and time that materials are in process. Before going any further, I would like to show you a film on arc characteristics of different manual rods and the flux cored CO_2 process. This film was prepared by Batelle Memorial Institute for National Cylinder Gas Co. and was taken at approximately 4,000 frames per second. What we are interested in is the penetration of the CO_2 process. Watch the arc dig into the plate. I will say at this point that no special technique is necessary to achieve this penetration other than to set the voltage and current at approximately the correct settings. (*At this point, speaker showed the film.*)

Now I will go into point one that I mentioned above. Tables 1 and 2 show weld material cost and arc times per inch of ¼-inch fillet.

The weld material cost includes wire, gas and flux in the cost of the metal deposited, not including labor and overhead. You will notice that the weld material costs for the flux cored process are about double 6012 rod costs and about 50% higher than 7018 weld rod which

TABLE 1

Process or Electrode	*Material Cost Per Inch of ¼-in. Fillet*
¼-in. E-6012	$0.0026
7/32-in. E-7018	0.0034
Bare wire—argon, 1/16-in.	0.0056
Bare wire—CO_2, 1/16-in.	0.0027
Flux cored wire, 3/32-in.	0.0050
Magnetic fluxed wire, 3/32-in.	0.0048

TABLE 2

Process or Electrode	*Arc Time (min.) Per Inch of ¼-in. Fillet*
¼-in. E-6012	0.110
7/32-in. E-7018	0.105
Bare wire—argon, 1/16-in.	0.065
Bare wire—CO_2, 1/16-in.	0.045
Flux cored wire, 3/32-in.	0.050
Magnetic fluxed wire, 3/32-in.	0.052

we use as a standard high grade weld rod. You will also notice that the speed is about double that of either of these rods. So we have twice as expensive metal laid down twice as fast. From this, taking a ¼-inch fillet, we find that manually we could lay down about 10 inches per minute or thirty feet per hour using 60% duty cycle. This comes out with a weld metal cost of about $1.26 per hour (7018) and if we take a welder at $2.00 per hour and put a 100% overhead on him this makes an hour's worth of welding by this man worth $5.26—no profit or General and Administrative expense. Dividing this by 30 feet, we get a cost per foot of $0.17. Using the same process of figuring costs and using a flux cored CO_2 process, which doubles the material cost per inch and also doubles our speed, we find our metal cost to be approximately $3.60 but our man is still the same man so we get a $7.60 cost per hour. Divided by 60 feet, this gives us about $0.12 a foot. This is considerably less expensive. Actually, we find a higher duty cycle possible because our man is not changing electrodes and he gets more than twice as many feet per hour. This demonstrates that you can use the process economically. Also I have used larger wire than 3/32-in., which increases the speed and is as much as two cents a pound cheaper. These larger wires give as much as 10% to 20% faster melt-off rate than the 14 lbs. per hour that Becker and Christopher report for 3/32-in. wire. (*Speaker then showed a series of slides from the Becker and Christopher paper noted above, demonstrating the greater penetration achieved by CO_2 processes.*)

I am sure that you will agree that the CO_2 process fillets are considerably stronger than the manually produced

* Gas Metal-arc Welding of Low Carbon Steels by G. Christopher and Becker, Vol. 39, Number 5, May 1960 Welding Journal.

Mr. Molthop is Vice-President, Vierling Steel Works, Chicago, Ill.

ones of the same sizes. Becker also shows in his paper that while speed and thickness of plate do affect the penetration, they do not affect it to the extent that one might expect. He also shows that the angle at which a fillet weld is made determines the depth of penetration, and makes allowances for this in his tables of equal fillet strength.

Becker and Christopher show the following equivalent fillet sizes:

E 70 XX Electrodes		*CO_2 Processes*
3/16	=	1/8
1/4	=	5/32
5/16	=	3/16
3/8	=	1/4
1/2	=	5/16
5/8	=	3/8

Everyone will agree that this shows a radical cost saving potential. In fact, a simple reduction of 1/16-inch in the usual fillet which we make will give us enough to effect at least a 50% saving in weld cost over manual welds. Your weld speed would be up to four times as fast and your metal cost would be about the same per inch as the manual rods. Similar savings result in butt and groove welds. No preparation is necessary for single pass welds from both sides in thicker plates than is possible manually. Also, smaller bevel angles are required in prepared plates, reducing the amount of weld metal to be deposited and hence raising the speed by which it is deposited. Along with the faster melt-off rate, this raises the speed considerably over manual joints.

Using a timer built into some models of these machines, it is possible to make burn-through spot welds to attach thin plates and sheets to a suitable backup. A louver plate was attached to an angle (Fig. 1) at the rate of 60 per hour and there was very little distortion due to welding. After attaching the plate to the angle a small natural gas torch applied to the bent flange removed the distortion due to welding and bending and removed any "oil canning" in these plates.

One problem in using this equipment is handling the water line, gas line and control box for the process. We solved this by mounting the equipment on a double hinged boom. This gives us a 45-ft. radius half circle for welding without dragging hoses and electrode cables around the floor. This is one of the things you must do to use this process efficiently.

I don't want to completely neglect other types of welding. Figure 2 shows a small shop girder builder. The generators (2-750 amp. units) were purchased to drive studs (up to 7/8-in.). These will also operate a twin wire automatic submerged arc welder. This welder positions itself in the girder, takes up little floor space when not in use and has been generally quite satisfactory.

With reference to assembly practices, I might say that we have been trying to save fitting and assembly time by tacking assemblies together and drilling from the

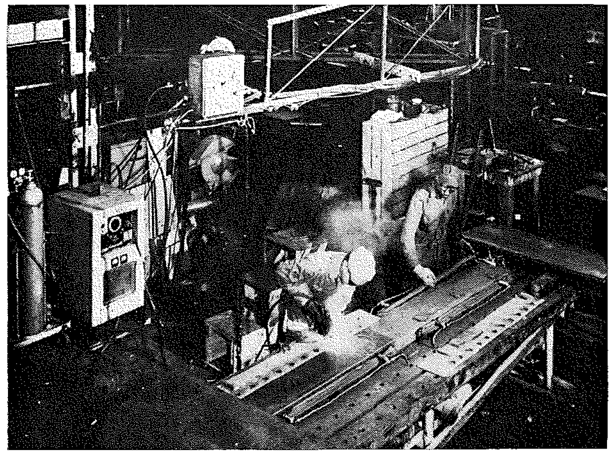

FIGURE 1

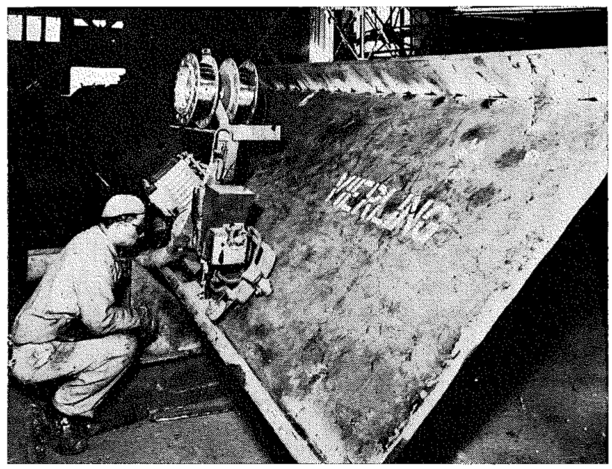

FIGURE 2

solid. One fabricator fitted some rather large girders together and drilled them with Buckeye automatic drills, which Mr. Kidd described at a previous conference. We now use these drills to drill from the solid the beam splices in continuous highway beam spans. They have led to considerable savings.

One cost savings in assembly could come from the CO_2 welding processes. That would be to use a fitter welder. If welding speed is increased enough it might become economical for a man to fit and weld, eliminating the extra handling from the conventional pick up, fit, put down, pick up, weld, and put down sequence.

In closing, I think we will have to do some research on the CO_2 processes to prove to the satisfaction of our customers that we can reduce these fillet sizes. Becker and Christopher are doing work on manufacturing products where they can take advantage of these savings today. We have to be more conservative. I would like to say that the cost of the necessary machinery to use these CO_2 processes is considerably less than the cost of many of the other machines described in this symposium.

Investment Policy

R. A. SHAW, SR.*

You have heard discussions today relative to the "What," "How," "When," and "Where" the projects should be done. I would like to discuss with you now the last step in this over-all discussion which consists of what results are anticipated if the capital expenditure is made. I believe that the keynote of the discussions that we have had is the need for adequate planning. This adequate planning also is important in determining the financial benefits that will accrue from capital expenditures in order to achieve the greatest return on the investments made.

It is my opinion that it would be beneficial to establish a working committee consisting of Sales, Operating, Engineering, Industrial Engineering, and Accounting personnel to completely study the various alternatives that might exist within given areas of a contemplated project. It is felt that such a group could develop the data and present to management all of the facts necessary to assure that the proper decision regarding any capital expenditure can be made.

The financial aspects involved in a contemplated capital expenditure, broadly speaking, are:

1. *The Sales Proceeds and Costs as They Exist Before the Contemplated Expenditure is Made.*

   This, in itself, could take various forms, for example, if the project contemplated an entirely new product line, it is conceivable that none of the current costs are applicable and, therefore, the base would become zero. If the project contemplates, however, extensions to existing product lines, then the base would include the current volume, sales, and costs properly applicable to the production and sales of this product.

2. *The Sales Proceeds and Costs as Contemplated After the Capital Expenditure has been Made.*

   This evaluation, of course, would include the contemplated volume, sales proceeds, and costs of manufacturing the particular products which would be produced and sold after the expenditure is made.

3. *Benefits*

   The difference between the foregoing Items 1 and 2 then becomes the financial benefits accruing from the capital expenditure after deducting Federal Income Taxes. The net profit then can be related to the total capital expenditure to determine the number of years (or per cent return) required to recover the investment.

4. Comparison, at a later date, of the costs and sales proceeds actually achieved with those anticipated (or claimed) for the project.

Before I illustrate some methods which can be used to present these financial facts to management for its decision regarding capital expenditures, I believe that it may be well to discuss the word "planning." In the area of capital expenditures, some projects are rather large in scope and others can be placed in the minor areas. It is also important that the "planning" segregate those projects which are of a desirable nature as opposed to those which are of a necessity nature. Let us assume that this working group which I mentioned would have information concerning all of the facilities now at hand including such data as year of acquisition, productive capacities, production rates, repair costs, and other key information which would permit an objective look as to when the particular facility should be replaced or completely rebuilt. By properly analyzing these data, a long-range program can be established for keeping the existing facilities or equipment in good workable condition or establishing a program for their replacement. The current year's program can then include those projects which management deems to be the most necessary. In the area of the desirable projects, it is also well to establish a long-range program which indicates the type of product contemplated to be sold, the market potential, facilities required, and so on. By having periodic reviews of these long-range programs, management can determine when and where these particular projects should become fact.

It may be desirable for this working group or some other group to prepare periodic reports on expenditures made on approved projects. In this way, then, management is informed of the cash outlay required from period to period. I would like to illustrate some methods of presenting these financial data to management.

Exhibit I

This exhibit has been prepared to illustrate a method of presenting the financial story to management. It contains a brief narrative of the proposed project and summarized financial data concerning the benefits expected.

Exhibit II

This exhibit illustrates in somewhat more detail the information relevant to the changes in the financial picture expected to result from the proposed project.

The data are detailed to show the benefits from the increased volume and the benefits accruing from cost

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Mr. Shaw is Vice-President, Operating, American Bridge Division, U. S. Steel Corp., Pittsburgh, Pa.

* *Mr. Shaw's paper was read by Mr. R. B. Waldon, Manager of Cost and Statistics, American Bridge Division, U. S. Steel Corp., Pittsburgh, Pa.*

XYZ COMPANY
Estimated Earnings and Return on Investment
Project No. 1401 - Additional Structural Fabricating Facilities
At No. 1 Plant

Project Expenditures

Investment chargeable to capital (depreciable property)	$ 450,000
Investment chargeable to expense	150,000
Total Project Expenditures	$ 600,000

Brief Statement of Proposal

In recent years the trend of our business has indicated that an increasing amount of light fabricated work has been assigned to the No. 1 Plant. Because this lighter work requires more floor area and the fact that the existing layout and cranes are not appropriate for this type of fabrication, congestion and operating ineffeciencies have resulted. It is proposed, therefore, to construct a new building for fabrication of light structural work including seven new pendant-controlled cranes; converting the seventeen existing cranes to pendant controls; relocate, install, and dismantle various facilities as required. In addition to the economies this new layout will provide, added capacity for light fabricated work will also result from this expenditure.

Summary of Benefits

It is anticipated that the gain in profits accruing from this expenditure will be $151,500 annually (after Federal Income Taxes). The investment will, therefore, be recovered in 3.5 years. (See Exhibit II for further detail.)

General

All financial data reflect current wage and price levels. It is expected that satisfactory operations will be achieved six months after completion.

EXHIBIT I

XYZ COMPANY
Estimated Earnings and Return on Investment
Project No. 1401 - Additional Structural Fabricating Facilities
At No. 1 Plant

Financial Changes (Annual Basis)

(Dollar Data in Thousands)

	Present	Proposed	Total Changes
Capacity - Net Tons	52,000	60,000	8,000
Expected Volume - Sales - Net Tons	50,000	55,000	5,000
Fabricating Sales Proceeds	$14,630	$16,093	$1,463
Fabricating Costs:			
Net Material	$ 6,500	$ 7,150	$ 650
Direct Labor	3,000	3,300	300
Overhead Costs	3,150	3,455	305
Depreciation - Present Facilities	100	100	-
Depreciation - Added Facilities	-	20	20
Selling & Administration Expense	550	605	55
Total Fabricating Costs	$13,300	$14,630	$1,330
Fabricating Profits	$ 1,330	$ 1,463	$ 133
Reduced Operating Costs:			
Direct Labor	$ 1,080	$ 970	$ 110
Overhead	590	530	60
Total	$ 1,670	$ 1,500	$ 170
Total Change Before Federal Income Taxes			$ 303.0
Federal Income Taxes at 50 per cent			151.5
Net Change After Federal Income Taxes			$ 151.5
Years to Recover Investment			3.5
Per Cent Return on Investment			28.9

EXHIBIT II

reduction. Behind these cost data, of course, would be more detail which would be useful, at a later date, in comparing the claimed benefits with the actual benefits.

There are many ways, and in varying detail, in which these financial data could be presented. For example; an exhibit could be prepared showing all of the equipment and facilities required, the present and proposed capital investment, data concerning the proposed products to be produced with expected selling prices, and so on.

Exhibit III

In this exhibit we have illustrated varying methods of showing recovery of investment, using the financial data from Exhibit II. No one method has been universally adopted among accounting organizations. These data are expressed as the number of years it will take to recover the investment as well as the per cent return.

Examples of Methods of Calculating Recoveries of Investment

A. Recovery of Total Investment (Capital and Expense)

1. Investment

Applicable to Capital		$450,000
Applicable to Expense		150,000
Total		$600,000
Total Annual Increased Profits After Federal Income Taxes		$151,500

2. Calculations of Return on Investment

Capital Investment		$450,000
Expense	$150,000	
Less Federal Income Taxes 50%	75,000	
Net Expense		75,000
Total		$525,000

Years to Recover $525,000 + $151,500 = 3.5 Years
Per Cent Return on Investment $151,500 + $525,000 = 28.9 Per Cent

B. Recovery of Capital Investment Only

Years to Recover $450,000 + $151,500 = 3.0 Years
Per Cent Return on Investment $151,500 + $450,000 + 33.7 Per Cent

C. Recovery of Total Investment (Capital and Expense) Based on Added Profit and Added Depreciation.

Total Added Profit After Federal Income Taxes			$151,500
Total Added Depreciation			20,000
Total Added Profit Plus Depreciation			$171,500
Capital Investment.		$450,000	
Expense Investment	$150,000		
Less Federal Income Taxes	75,000	75,000	
Total			$525,000

Years to Recover Investment $525,000 + $171,500 = 3.1 Years
Per Cent Return on Investment $171,500 + $525,000 = 32.7 Per Cent

D. Return on Initial and Average Investment

1. Capital Only

Annual Income After Federal Income Taxes $151,500
Capital Investment $450,000
Return on Initial Investment $151,500 + $450,000 = 33.7 Per Cent
Return on Average Investment $151,500 + $225,000 = 67.3 Per Cent
(i.e., $450,000 + 2)

2. Total Investment

Annual Income After Federal Income Taxes $151,500
Total Investment $600,000
Return on Initial Investment $151,500 + $600,000 = 25.3 Per Cent
Return on Average Investment $151,500 + $300,000 = 50.5 Per Cent
(i.e., $600,000 + 2)

EXHIBIT III

1. *Method A*—portrays the recovery of total capital and expense investment after deducting the Federal Income Tax applicable to the expense portion. This approach has the advantage of presenting the total investment requirements to management. Where the amount of expensed charges is large relative to capitalizable costs, failure to include expensed items in the investment base may cause serious overstatement of the rate of return. As a result, management might unknowingly approve a project from which cost cannot be recovered.
2. *Method B*—presents the recovery of investment for the capitalized portion only. This method becomes a disadvantage for the reason stated under Method A.
3. *Method C*—some companies have used the added profit plus added depreciation method to determine the recovery of investment. This is sometimes referred to as the "Available Cash" method. Depreciation becomes a non-cash cost during the life of the investment.

Therefore, the added profit plus the added depreciation applicable to the project increases the cash availability.

4. *Method D*—portrays another way to express the return on investment. The first alternate shows the recovery of the capital investment only on an initial basis and average basis. The second alternate shows the recovery of the total investment on an initial basis and average basis. Return on the average investment is obtained by dividing the initial investment by two.

The Discounted Cash Flow Method

Under the discounted cash flow method, the rate of return is computed on the amount of capital unrecovered from period to period instead of upon the initial average amount invested. This rate of return may be defined as the maximum rate of interest that could be paid for the capital employed over the life of an investment without loss on the project. This method of presentation is somewhat more complicated than the other methods mentioned, though it has many advantages.

Exhibit IV

Quite frequently, management becomes vitally interested in obtaining information concerning the progress made on approved projects which are fully operative.

XYZ COMPANY
Comparison of Proposed Versus Actual Earnings
Pnoject No. 1401 - Additional Structural Fabricating Facilities
Period Covered - Year 1960

(Dollar Data in Thousands)

	Proposed	Actual
Expenditure - Property	$450	$442
Expense	150	146
Total	$600	$588
Benefits Before Federal Income Taxes	$303.0	$380.0
Benefits After Federal Income Taxes	$151.5	$190.0
Years to Recover	3.5	2.7
Per Cent Return	28.9	36.9

Analysis of Financial Changes Before Federal Income Taxes

Original Expected Annual Financial Changes	$303
Actual Annual Financial Changes	380
Financial Changes Attained More Than Anticipated	$ 77

	Statement of Proposal: Present (1)	Statement of Proposal: Proposed (2)	Year 1960 Results: Actual Tons at Proposed Sales and Costs Levels (3)	Year 1960 Results: Actual (4)	Year 1960 Results: Difference (5)
Tons Produced	50,000	55,000	57,000	57,000	
Sales Proceeds	$14,630	$16,093	$16,678	$16,530	$148 *
Fabricating Costs					
Material	$ 6,500	$ 7,150	$ 7,410	$ 7,296	$114
Labor	3,000	3,300	3,420	3,363	57
Overhead	3,150	3,455	3,580	3,580	-
Depreciation	100	120	124	124	-
Selling and Adm.	550	605	627	627	-
Total Cost	$13,300	$14,630	$15,161	$14,990	$171
Net Profit	$ 1,330	$ 1,463	$ 1,517	$ 1,540	$ 23
Profit Change	$133	$54	$23		
Operating Costs					
Labor	$ 1,080	$ 970	$ 970	$ 970	-
Overhead	590	530	530	530	-
Total	$ 1,670	$ 1,500	$ 1,500	$ 1,500	-
Cost Change	$170	-	-		
Total Change	$303	$54	$23		$ 23

Major Factors Responsible for Financial Changes

$ 54 - Volume Effect - Fabrication of 57,000 tons of product as compared to anticipated 55,000 tons resulted in this increase in profits, as exhibited between Columns 2 and 3.

$148 * - Sales Price - Actual per cent markup less than anticipated due to effect of current competitive prices.

$114 - Material - Principally due to lower than normal scrap yield resulting from ordering material cut to length needed for contracts.

$ 57 - Shop Performance - Primarily due to favorable labor performance on all products produced during period.

EXHIBIT IV

For that reason, a simplified example of a report showing the benefits claimed for the project as compared to the actual results for the period under study is presented as Exhibit IV.

Depending on the complexity of the operations involved, the inherent operating problems connected with these operations, etc., determine the course and detail the results would be displayed. The length of the period for which individual project reports would be made is also dependent on the benefits actually achieved.

Inflationary Effect of Cost of Facilities on Profit and Loss

Some companies because of the nature of their business must be heavily invested in long-term facilities. The real capital of such companies is eroded away by the present corporate income tax. This is because it takes many more of today's dollars to replace equipment as it wears out than it took originally to purchase the equipment many years ago. But depreciation allowed in computing taxable income is limited to the smaller number of dollars originally expended. The difference between such depreciation and the larger amount needed to recover the purchasing power expended—is arbitrarily called income and taxed as such. The corporate income tax not only hampers the formation of new capital but also erodes away existing capital, and does it in highly inequitable fashion because of the varying importance of depreciation to the businesses taxed. Permit me to give you a simplified example of this problem.

Suppose in 1940, capital expenditures were made at a plant amounting to $100,000, and the life of these facilities is twenty years. In 1960 we will have recovered the $100,000 through cost. However, to replace these facilities in 1960, according to the *Engineering News Record,* it would cost $330,000 (1959 = 100%; 1940 = 329.5%). This represents an increase of $230,000 cash required to install these facilities which must come either from profits or by floating a loan. At present interest rates of around five per cent, you can see that this just makes matters worse, as the interest costs must also be recovered from profits. It appears to be a never-ending spiral and lends credence to the importance of good, sound planning of capital expenditures. It is my firm belief that legislative correction of the tax injustice or the renewal of granting of the equivalent of Certificates of Necessity should be made. If enough companies and individuals would make these thoughts known to their legislative representatives, I am sure that the proper corrective action would be made in our tax laws.

Gentlemen, I trust the foregoing comments have been helpful to you in your various planning functions. I personally feel that good planning of investments is sound business and that adequate financial reporting on the project plans assists management in making the proper decisions.

Discussion on Symposium

(*Q*) H. W. BRINKMAN, Phoenix Bridge Co.: Are there any codes or acceptable standards which establish the sizes of "deep" welds which can be substituted for given sizes of standard welds?

(*A*) MOLTHOP: As far as I know, there are none. That is one of the reasons I discussed this subject in my presentation. I believe that, if there is enough interest, we can get something done in this area. We can begin by asking for a very conservative increment, and I believe we can demonstrate quite easily that it is conservative. Later, we may be able to go further and achieve greater benefits.

* * *

(*Q*) J. A. RAU, Allison Steel Manufacturing Co.: I would like to discuss the automation that was illustrated by the drilling of the beam and column work. This indicates a trend away from welded construction in the shop. In our plant we are currently thinking strongly about an increase in welded shop fabrication, rather than the type of fabrication shown here. I would like to have the panel's thinking about the trend in competitive methods of fabrication.

(*A*) DULL: I do not believe anyone can be certain of trends in fabricating methods. International Steel has always been a booster of welding, and still is. We have approximately 150 pieces of welding equipment, and have never had less than sixty welders in our shop. Under normal conditions we will take any kind of work that fits our facilities. It is not our plan to push riveting or bolting and to take any emphasis off welding. We believe in welding and will continue to promote it.

(*A*) KIDD: Many times the design of a particular job dictates the way in which it will be fabricated. Often welding is more easily adapted to that particular design. However, we feel that there will still be many jobs for which high strength bolting, with proper automated equipment, will be more economical than welding.

* * *

(*Q*) M. H. KING, Carolina Steel Corp.: What is the minimum amount of duplication necessary to justify the automated equipment that has been discussed?

(*A*) HALL: In order for the equipment to pay off, I'd say in the case of a bulldozer—if it were used four hours per day, you would get your money back. An angle roll is usually more expensive than a bulldozer, so it should be operated four or five hours a day. In the case of a plate roll of 2½-inch capacity, you should probably operate that equipment six to eight hours a day. The C-frame press which you saw would cost ninety thousand dollars today, and you would need to operate it a minimum of eight hours per day.

* * *

(*Q*) S. KAPELSOHN, Grand Iron Works: What is the approximate cost of the programming method?

(*A*) KIDD: Approximately a quarter of a million dollars.

* * *

(*Q*) A. E. SHLAGER, Groisser & Schlager Iron Works: I have two questions. First, on beam work, how do you establish your working points? Most beam flanges vary somewhat, which must have some effect on layout in an automatic operation. Second, how do you prevent the point of the drill from "walking"?

(*A*) DULL: With reference to the first question, we drill column or beam flanges about the center of the web, exactly as they would be laid out or punched by other methods. The variations are taken up by the operator compensating for the off-center conditions that do happen consistently. We started out doing it with a rule, measuring and then checking periodically as we run through a section. We are now installing a web-finding device on the machine. Hydraulically operated "finders," push-button controlled by the operator from the operating station, come down to the web of the beam. A preset dial on this device shows the relationship of the center line of the beam to the datum line (or roll line). The operator reads this dial and compensates for variations by adjusting the drill itself. These adjustments take only a few seconds.

Answering your second question, rigid clamping prevents drill "walk," and is the secret to long drill life. The material being drilled does not "float" as it does under a typical radial or other standard type of drill machine. It is the movement of the material, not the cutting of the steel, that wears out the drill in the average shop.

FRIDAY MORNING SESSION

May 6, 1960

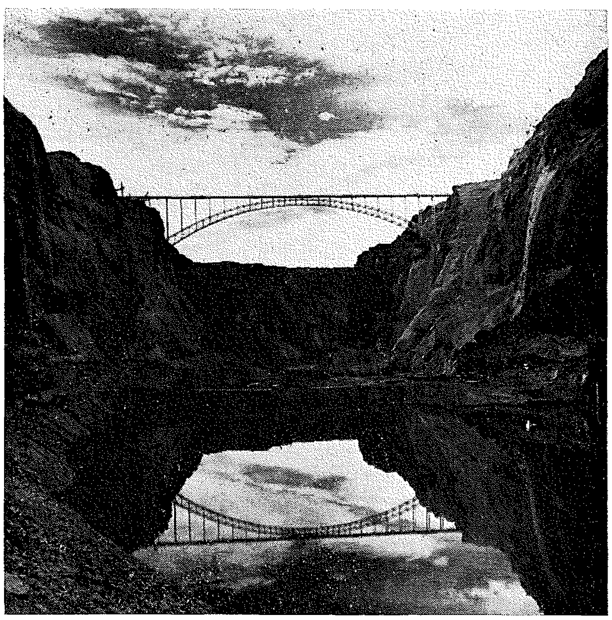
Presiding: D. F. Guell, Chief Engineer, Denver Steel & Iron Works Co., Denver, Colo.

Color Film: Crossing at Glen Canyon

The motion picture, describing the construction of the Glen Canyon Bridge across the Colorado River, was presented through the courtesy of the Judson Pacific-Murphy Corporation, Division of Yuba Consolidated Industries, Inc., Emeryville, Calif., fabricators and erectors of the bridge. Mr. J. Philip Murphy, president of the Judson Pacific-Murphy Corporation, introduced the film.

Glen Canyon Bridge—Highlights on Design, Fabrication and Erection

ROBERT SAILER



When Glen Canyon Dam, a $330 million project, was in the overall planning stage, where and how to cross the Colorado River was studied extensively. The crossing was the missing link for the access road from Arizona, on one side of the river, and from Utah on the other side. At first, consideration was given to routing the highway over the dam. The crest of the dam is over 100 feet below the rim of the canyon. To build highway approaches to fit the curved crest of the arch dam and meeting present day specifications for curvatures, sight distances and grades would have required a 90-foot deep rock cut on one side and a tunnel on the other. Of major objection to a crossing on the dam was that the two new highways leading to the dam could not be connected until the dam was completed.

Realizing that a vehicular crossing over the canyon for use during construction would be of tremendous advantage, cost estimates for a suspension bridge serving construction purposes were made. A bridge with 24-foot roadway and 2-foot safety curbs on each side was estimated to cost $1,800,000.

When the State of Arizona and the Bureau of Public Roads expressed their willingness to participate in the cost of a permanent crossing approximately 1,000 feet downstream from the dam, all factors previously mentioned were reconsidered. It was concluded that a permanent bridge downstream from the dam, and constructed at the earliest possible date was the best solution.

For the permanent crossing an arch bridge and a suspension bridge were considered. The cost of the arch was slightly less than that of the suspension bridge. Further, the choice to use the arch span offered greater flexibility in planning the road approaches and vista areas and was much more pleasing in appearance.

In the estimates above, a fixed arch having solid ribs similar to the Rainbow Arch bridge at Niagara Falls was used. The Rainbow Arch is about 70 feet shorter than the Glen Canyon Bridge. As further studies were made, it became apparent that a solid rib arch, which has a large exposed wind area, would seriously effect the stability of the Glen Canyon bridge, as the overall width is only 40 feet. By comparison, the Rainbow bridge is 60

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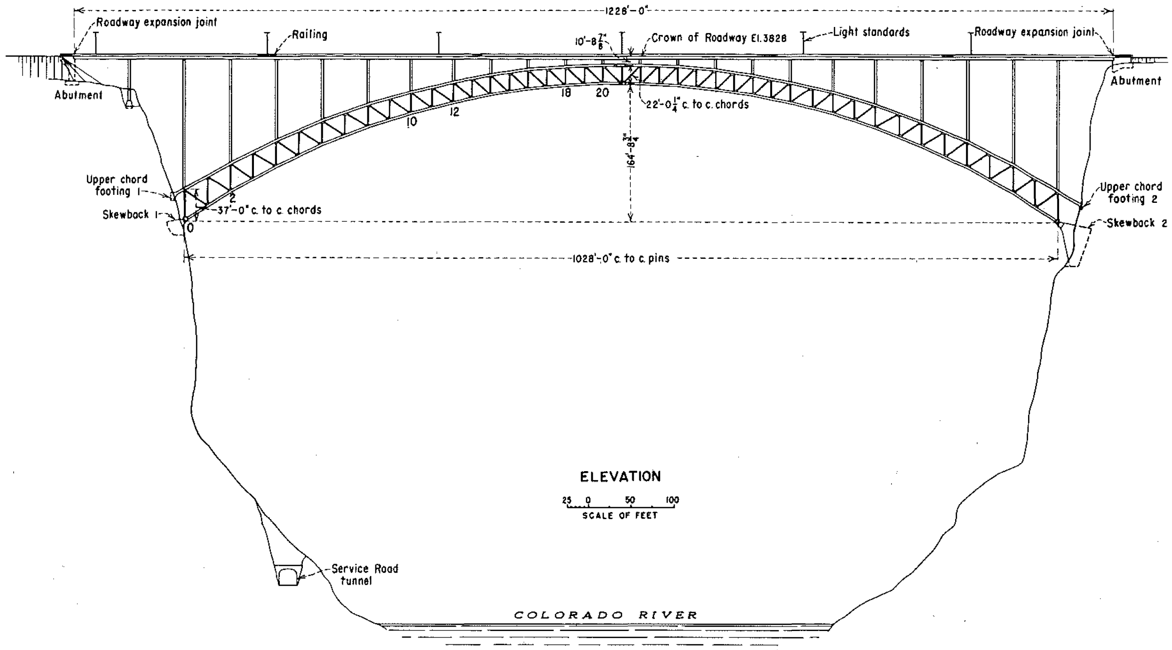


FIGURE 1

feet wide. Truss type arches were then studied in more detail. Types investigated were: fixed arch, two-hinge arch with pins in lower chord and midway between lower and upper chord, and a 3-hinge arch for dead load, and 2-hinge effect for live load. The hinged arches were found to have decided weight advantages over the fixed arch. Also, the fixed arch is more difficult to erect and hence the erection costs are likely to be higher.

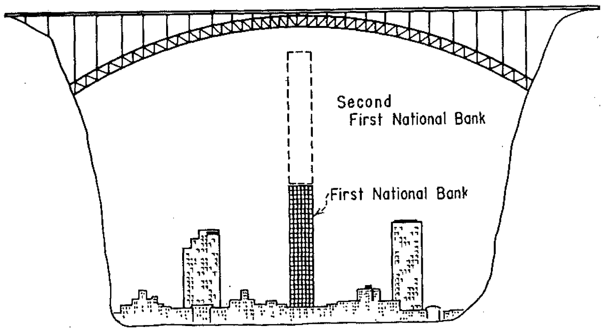
The final choice was a two-hinge arch with hinges in the bottom chord. Inaccuracies in control during the cantilever erection will cause only small stress uncertainties. Also, the heavy top and bottom chords of the arch truss were more uniform in cross section than for any other type of arch. This makes for simpler details and easier fabrication. Figure 1 shows the final design. The overall length of the bridge is 1,228 feet. The arch has a span of 1,028 feet, and is the fourth longest in the world, the second in the United States. The rise is 165 feet or 1/6 of the span. The bridge deck is 700 feet above the Colorado River. Well just how high is 700 feet? Figure 2 shows the bridge superimposed on the skyline of Denver. The tall building is Denver's tallest, the First National Bank Building. Put another one on top of it, and you just about reach the bridge.

Figure 3 shows the typical cross-section. The bridge has a 30-foot roadway and two 4-foot sidewalks. For stability reasons the trusses were spaced 40 feet apart. The half section to your right shows the section at the crown, with the floor beams supported directly on the truss. The half section to your left shows the section at the quarter point. Here the floor beam is supported by columns. All columns are box sections. The column at the skewbacks is 156 feet long, 38 inches wide, and 30 inches deep. Manholes provide access to the inside and ladders were provided full height.

The stringers bear on top of the floor beams and are continuous beams. This resulted in a considerable saving over simple beams. The bridge deck is a 6-inch thick reinforced concrete slab.

In the trusses, two kinds of steel were used. Diagonals and verticals are A7 carbon steel. They are relatively light, and are made up of four angles and plates. For the chords, a manganese silicon steel was used, which is suitable for riveted work. It had been used on several recent monumental bridges and was recommended by leading steel companies. For 3/4-inch material, the ultimate strength is 72,000 psi, and the yield strength 50,000 psi. For thicker materials, the strengths are slightly less. The design stresses were 24,000 psi in tension and approximately 19,500 psi in compression. All truss chords were box sections with 30-inch webs, and 30-inch cover plates. Solid rather than perforated plates were used for maximum efficiency of material. The heaviest member was at the skewback and had a cross sectional area of 302 square inches. The web thickness was 4 inches. The cover plate 7/8-inch thick.

In view of the heavy material, riveted rather than welded work was specified. In an arch truss most of the material is in compression, and the advantages of welded work as far as net section is concerned becomes unimportant. Welding was used for the bracing members, in the stringers and diaphragms for columns and truss chords. For reason of heavy thickness field riveting was required for the arch trusses and bracing. High tension



SKYLINE OF DENVER
FROM 6TH AVE.

FIGURE 2

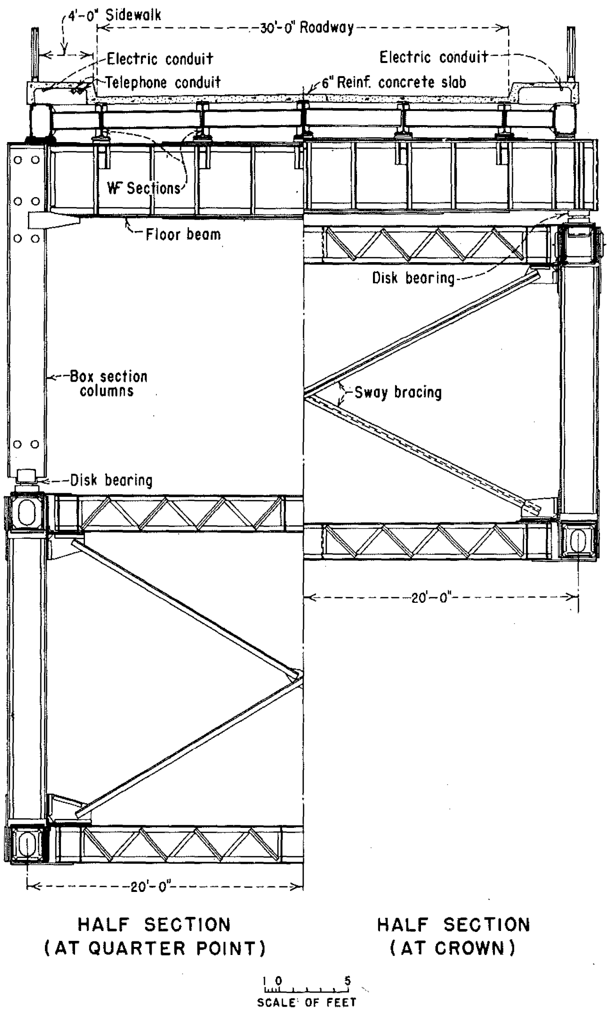
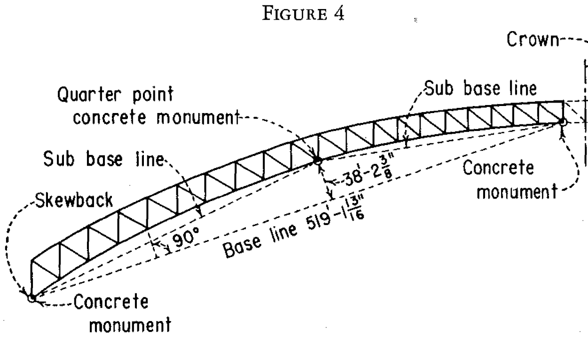


FIGURE 3

bolts were used for the field connections in the floor system.

The bridge contains 3,920 tons of structural steel. Of this, 1,590 tons or 40 percent is high strength manganese-silicon steel.

FIGURE 4



CONTROLS FOR YARD ASSEMBLY

FIGURE 5

The structural steel was fabricated at the plant of Judson Pacific-Murphy at Emeryville, California. The work was performed to a high degree of workmanship. Of particular interest was the yard assembly of each half of the arch truss. This was required by the specifications to assure proper fitting in the field. Each half of the arch truss was over 500 feet long. Figure 4 shows one of the halves completely assembled. Comparing sizes of car and men gives a perspective of the size of the arch. Figure 5 shows the method used for control of length and rise. A base line accurately surveyed, with steel pins set in concrete monuments was established. Concrete monuments were surrounded with steel posts as a precaution against damage from trucks and equipment used in assembling. From the base line the location of each panel point was determined by ordinates and abscissas. As the work progressed, it was found that this work could be speeded up by using sub-base lines to the quarter point and then measure from there. The truss members were assembled and made up 25 percent with a combination of drift pins and bolts. The truss chords were assembled with particular care to assure that the milled ends were in contact all around. The first yard-assembled truss was completely assembled and checked out for dimensions from skewback to crown. While so assembled the connections were reamed full size.

While the reaming was still going on at the crown higher strength but when these steels were welded, the welding heat would wash out the heat treatment and the heat affected zone would usually become brittle. This naturally led into a new line of research to develop a steel which would retain its strength or even gain strength under the rapid cooling experienced adjacent to a weld. Steels are now available which meet this requirement. The rapid cooling after the welding arc has passed actually is a beneficial treatment and as far as this parent metal is concerned, the faster it cools, the better its physical qualities. Unfortunately, however, the weld metal usually does not have similar capability. Hence, some post heating to control grain growth and chilling is often necessary when the plate thicknesses differ greatly.

High-strength steel is more expensive than ordinary A7 structural steel. The designer must, therefore, get an extra value from the high-strength in order to make it an economical proposition. He can't afford to waste steel for splices and lap joints. By welding, plates may be joined edge to edge to make box and plate girders without the need for flange angles, splice plates and thousands of rivets. This is a real saving. Add to this the advantages of the use of high-strength steel and the savings become very worth-while. More important though, welding makes possible an entirely new system of design into which high-strength steel fits admirably.

Now, lest you think this is merely a theoretical discussion on the use of high-strength steels, let me show you a few things that we in California have done to utilize them thus far.

Twenty-five years ago, the steel companies brought out their alloy steels in structural shapes. California started using them soon after. In 1939, two rather large truss bridges were designed utilizing over 2,200 tons of this new steel. Two years later this steel was covered by ASTM tentative specification as "A242 Alloy Steel." In the intervening years many thousand tons of this alloy steel have been used in California highway structures. Well over 100,000 tons of A242 steel have been used in the last 10 years in welded structures alone. The next step into an even higher stress grade was therefore viewed with full realization of its probable economic benefits.

About 1953, when the early designs for the second Carquinez Straits Bridge were beginning to take shape, there had been no extensive use of the new 90,000 psi-yield steel in any application. The saving in weight and the smaller members which this steel would make possible were very attractive to the designer. We made a very thorough investigation of this new steel. Facts were collected and many discussions were held with various steel experts and metallurgists to prove its reliability for a structural use. Over 3,000 pounds of the new steel were tested, welded, broken and torn in the California State Highway Laboratory to discover its vital characteristics. Finally in 1954 it was determined that a substantial portion of the proposed Carquinez Bridge would be fabricated from the new 90,000 psi-yield steel.

The Carquinez Bridge just north of San Francisco is a double cantilever structure with two 1,100-foot main spans (Fig. 1). It closely parallels an existing structure built in 1927 and, for reasons of appearance, the new bridge was given the same general overall lines and spans.

With its all welded construction and bolted field connections, we see the clean lines and the trim members which welding makes possible (Fig. 2).

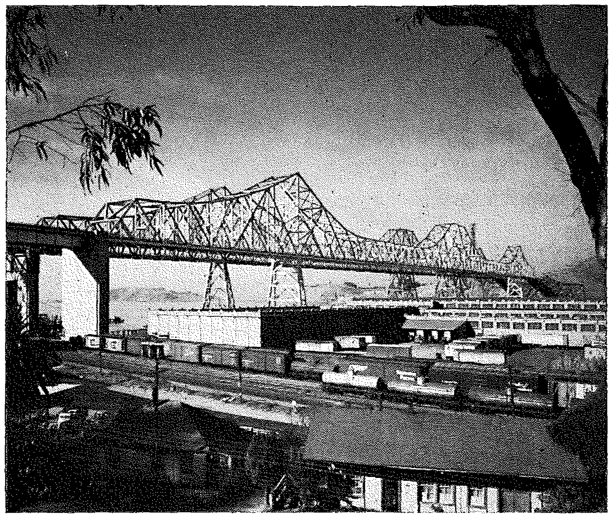


FIGURE 1

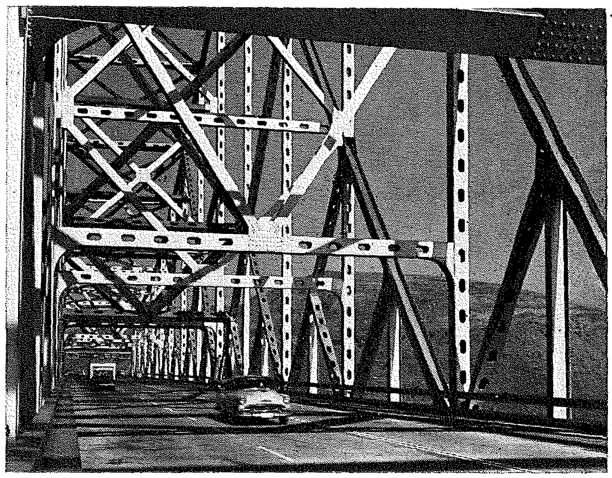
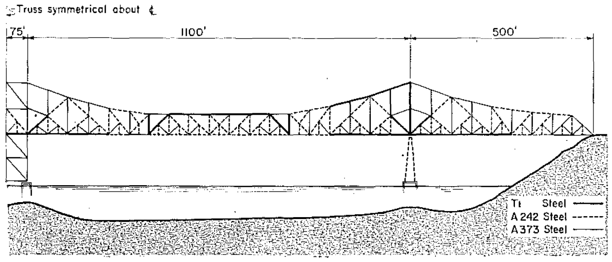


FIGURE 2

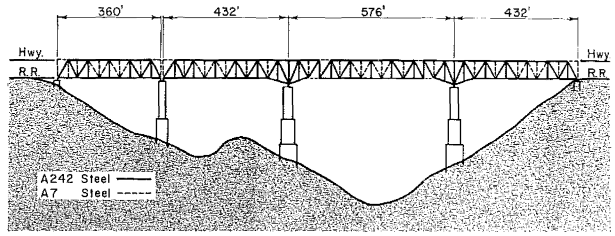
The heavily stressed members are made of T-1 steel, intermediate members of A242 steel and those lightly stressed were of A373 steel (Fig. 3). The total tonnage of these three types of steel is roughly 3,000 (2,910) tons of T-1 steel, 5,000 (5,370) tons of A242 steel, and 6,000 (6,440) tons of A373 steel. We figure that the use of alloy steels made possible a saving of about $800,000 in the cost of the superstructure.

Another large bridge using a combination of alloy steel



CARQUINEZ STRAIT BRIDGE

FIGURE 3



RAILROAD AND HIGHWAY BRIDGE
(WEST BRANCH OF FEATHER RIVER)

FIGURE 5

is the West Branch of the Feather River Bridge now under construction as a part of the Oroville Dam project in northern California (Fig. 4). Since the bridge carries a railroad on the lower deck and highway on the upper deck, the truss design features were governed by the railroad requirements. This necessitated riveted or bolted connections throughout and the welding of main members had to be confined to the strictly highway portions of the structure (Fig. 5). The use of high-strength steel was limited to A242. Although this bridge is a strictly conventional approach to the steel design problem, being mainly riveted and bolted, it illustrates the intermingling of A7 and A242 steel to achieve some very real savings in weight and size of members.

The Benicia-Martinez Bridge (Fig. 6) is now being constructed across Carquinez Strait about four miles above Carquinez Bridge. Like Carquinez it is all welded construction with bolted field connections. It makes maximum use of the best qualities of all three grades of steel.

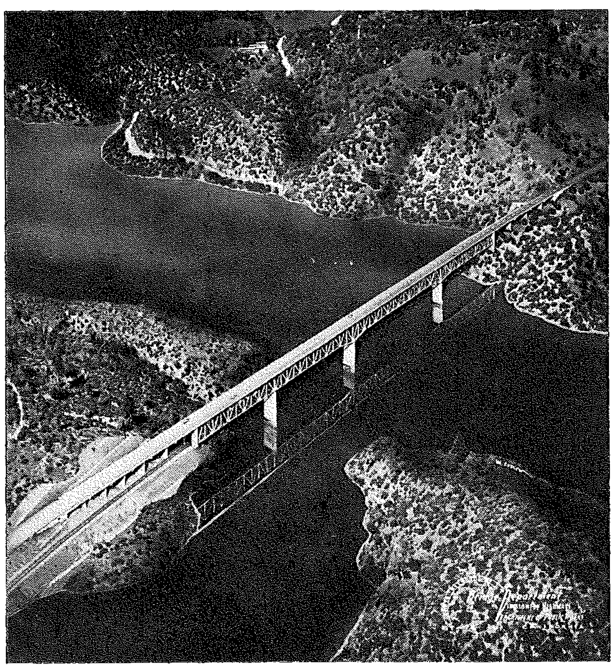


FIGURE 4

High-strength steel is used in the most highly stressed members (Fig. 7). It was also used in the joints to achieve the greatest strength with the least weight and bulk. The use of the high-strength material made it possible to economically carry a uniform depth of truss clear across the bridge without the necessity of deepening over the piers.

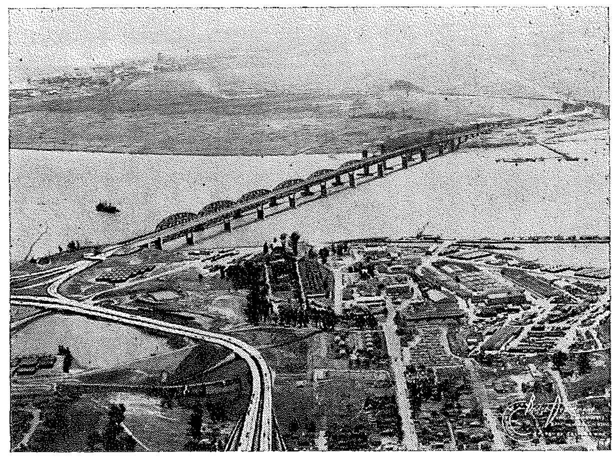
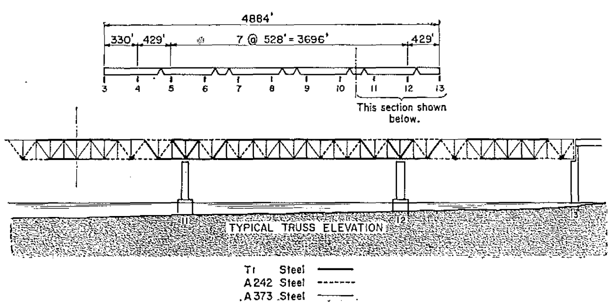


FIGURE 6



BENICIA - MARTINEZ BRIDGE

FIGURE 7

An interesting combination of high-strength steel which takes best advantages of the high-strength characteristics by means of welding, is to be found in the Whiskey Creek Bridge now being built in northern California (Fig. 8). A dam, now under construction, will flood out U. S. 299

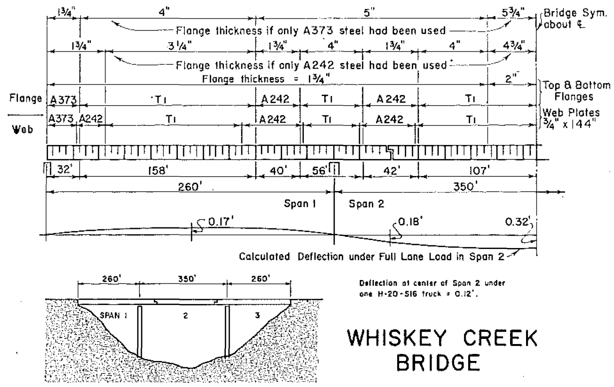


FIGURE 8

just west of Redding. The new line, high on a mountain side, crosses a deep canyon at Whiskey Creek. A three span welded steel plate girder has been designed for this location. The two end spans are 260 feet and the center span is 350 feet, which includes a suspended span of 260 feet. The web plate is maintained at a constant depth of 144 inches throughout the length of the bridge. In order to maintain this constant depth, three different stress grades of steel were used. T-1 steel is used in the web and flanges in the most highly stressed sections. This is graduated down to A242 steel in the areas of intermediate stress. And A373 is used in the areas where the stress is least. Another fact which should gladden the heart of a fabricator is that except for 90 feet in the middle of the long span, where 2 inch material was used, the flanges are a constant 1¾ inches in thickness clear across the bridge. Where this change was made to 2 inches, the additional thickness was taken on the outside so as to make the weld between flange and web continuously straight.

There are numerous advantages to this arrangement. There is of course first a savings in gross weight—both by reason of the welding and use of the high-strength steel. Very important advantages come from the fabrication simplification and also from the improved appearance. Most attractive though, is the economy to be found in the use of high-strength steel. In other words, the strength of the steel rises at a faster rate than does its price so that in high-strength steel you get more for your money.

In some cases the use of high-strength steel makes certain types of fabrication possible which would not have been possible had ordinary A7 steel been used. On Carquinez Bridge, it would have been impossible to have fabricated the trusses by welding A7 steel. The plate thicknesses would have been such as to make welded connections very difficult. The Whiskey Creek girders would have been much deeper and of variable depth had they been of A7 steel. These disadvantages also have an economical effect even though they cannot be evaluated by comparing prices.

So much for these notable examples of how we intermixed grades of high-strength steel within one structure. The question that most of you are asking, and do ask when you come into the office, is: "When will we get a chance to build another one?" Some of your representatives and members who are charged with promoting more fabricated steel emit cries of pain every time they see a concrete bridge being built. They keep up a continual pressure for more steel jobs. Unfortunately they are not too impressed by the fact that a steel job is often not the most economical, but they still persist in demanding more steel jobs.

Longfellow said that a man must be either a hammer or an anvil. But he overlooked the fellow who simply bellows.

Steel girders are generally not in competition for the short spans. By short spans in this case I mean those under 100 feet. A few cases will illustrate the point.

We made a comparison of concrete box girders, concrete tee-beams, and steel girders for similar bridge spans. We build a lot of separation structures, that is bridges over highways and freeways. These often have four spans, two long spans in the center and two short side spans. A common type on which we made the study had two 70-foot spans in the middle and a 40-foot span on each end. The concrete tee beam bridges came out cheapest at about $4.50 a square foot for the superstructure only. Concrete box girders were about 10% more expensive at a little over $5.00 per square foot. Steel plate girders were the highest of the three, another 15% higher or about $5.70 per square foot. These figures were all for superstructure only, to eliminate the variance in cost of substructure. However, taking into account the substructure, the total cost of these bridges per square foot averaged $8.20 for tee beams, $8.35 for box girders, and $9.59 per square foot for steel plate girders.

Another comparison was made of bridges with 100 foot spans over waterways. We compared the three types: concrete box girder, concrete prestressed girders, and steel plate girders. Just to keep the record straight we had only two examples of the prestressed girder, whereas we had seven concrete box girders in this category and six steel plate girders. The concrete box girder design was the cheapest with an average price of superstructure only of $6.34 per square foot. The prestressed bridges were slightly higher with a superstructure cost of $6.44 per square foot, whereas the steel plate girders came out highest with a cost of $8.17 per square foot.

It has been our experience, that regardless of location, comparable structures of the four different designs always seem to cost in about the same relationship. That is, tee beams or box girders are cheapest, then prestressed and finally steel at the top. Until you get well over 100 feet in span length, the steel bridges don't seem to be able to compete unless there are some other factors such as ease of erection or traffic safety, to allow one to disregard economics.

All of this is fine, but it doesn't answer the problem. Your people come in wanting more steel bridges. I point out the price is too high. They throw up their hands and say they have no control over price. Is that the end of the story?

If it is true that price is beyond your control, then you should strive as an organization or as an industry to make steel more attractive on other grounds. Attractive from any angle you can devise. Money is not always everything. Things like availability, time of delivery, safety and convenience, ease of erection and appearance—these all have an influence that will sometimes overrule economy.

I know when I speak of price and selling steel, some are going to say I'm talking to the wrong group. These are engineers, not salesmen. That is true. The steel industry has no dearth of salesmen out ringing doorbells. This message is not for them. The future of the steel industry lies in its management and its engineers who will find better ways to produce and fabricate steel and widen the range of its application.

Although many of you have varied interests in the field of construction, most of you are concerned with the fabrication of structural steel. I am told that under even the most optimistic interpretation, structural steel for highway bridges amounts to only 2 or 3% of the gross volume of the steel industry. Even with the other fabricating interests you have, your use of steel probably represents not more than 5% of the entire industry. Therefore, we share a minority position. Sitting at the foot of the table as we do, we do not carry a great deal of force or influence with the industry.

Although I may tell you, as fabricators in the steel industry, that your product is priced too high, there is not a great deal you can do about it. Every time the steel producers raise their price, you find a little more of your market slipping away.

Beside price, there is another factor that is stealing business from you. That is competitive materials. I do not need to tell you that concrete has cost you a lot of business. You are well aware of that. But are you aware of the inroads which other materials, aluminum, plastics, rubber, and even timber are making into your market? Are you aware of the very active and aggressive battle that these other materials are waging to supplant steel as a preferred building material? And even more important, what are you doing to fight back?

In California we have what we call a barrier rail which we use on almost all of our structures. It has a heavy concrete mass which will stop a car from going over the side of a bridge. It is topped by a metal railing consisting of a 5 inch pipe supported on stubby posts. In the space of about five years, that type of railing has gone from all steel to all aluminum. Unless somebody on the steel side of the picture gets interested, there is an item that is permanently lost to steel.

The aluminum people are frequent visitors to our offices seeking avenues through which to make further inroads into the steel dominance of the market. I suggest that people interested in promoting steel should show a similar ambitious interest.

I should not have to tell you other fields in which inroads are being made. No doubt each one is all too familiar to you. In the materials we use in highways, you are no doubt well aware that aluminum chain-link fence is rapidly becoming competitive. Aluminum plate guard rail is already on the market. Many signs and sign standards are already made of aluminum or plastics, while other materials too are squeezing into this field to edge out steel. Even rubber, a lowly slab of rubber, gives promise of substituting very adequately for that gold-plated steel detail on our structures—the expansion bearing.

Is it impertinent to ask—what are you doing about it?

You can do three things, at least. You can improve your own attitude; you can improve your facilities; and you can work collectively to improve the lot of the steel industry as a whole.

I have a personal note I'd like to add. I have heard it said that I am against steel. Nothing could be farther from the truth. To me a steel bridge typifies the fullest exemplification of bridge building. Steel made possible those long spans which have given bridge building its romance. I still relive the thrill of my first steel job every time I smell burned red lead paint. Because I do recognize and respect steel as the ultimate bridge building material, I am the more pained to see the industry sink into the morass of complacency. For these reasons, I feel compelled to make these few suggestions.

We still find fabricators who ask us to put out riveted designs because they do not want to convert their facilities to a welded operation. Riveted designs—when it is well established that they are anywhere up to 20% more expensive. This is about the same as requesting us to provide a lane on our freeways for horse-drawn vehicles. You may contrast this with fabricators in our area who are installing the latest equipment, who are actually putting a degree of automation in their operations. Automation, which can reduce that most expensive commodity—hand labor. Hand labor seems to be the hallmark of steel fabricating shop practice. Our structures are practically hand made in many cases by the same methods that were used 30 or 40 years ago.

The inevitable force of competition will in the end force you to move—to improve your plants, change your attitudes, and accept the new ways of doing things.

A newspaper reporter went to interview an old man on his hundredth birthday: "Pop, I'll bet you've seen plenty of changes around these parts." "Yep. And I've been agin every durn one of 'em." Believe it or not, there are fabricators like that.

The prevalent use of many different steels brings up another matter you should be interested in—

While the advent of these many superior alloy steels has opened up new vistas of interesting and economical application to the designer it has also opened a Pandora's box of troubles and confusion in specifications. In the interest of the industry as a whole and from the standpoint of simplicity of your own operations therefore, I would think it well for your organization to throw its weight behind some standardization in grades of steel.

Each of the steel companies has several steels in different stress ranges which they are pushing commercially. They can provide a steel with practically any given ultimate strength from 60,000 psi to over 100,000 psi and they are ready to go higher in the future. Now, where does this leave the designer? It would be theoretically possible to intermix steel in a structure so that some might be working at 20,000 psi, other at 30,000 psi, some other alloy at 35,000 psi, and so on. Theoretically this could be done, but as you all know it would be a mixed up mess to fabricate.

For purposes of orderly design and specification, and reasonable control during fabrication, it is essential that the stress grades of steel be grouped into classes. There should probably be not more than four classes to cover the range of available stresses.

Incidentally, who is in a better position than AISC to develop a good standard method of marking and identifying these different types of steels? As fabricators, you know the difficulties and hazards that come of having a number of different types of steel lying around the shop.

Heretofore, structural steels have been grouped into two principal grades: A7 and A242 using the ASTM designation. Recently, the very high-strength alloy steel was added as a third grade. A7 and A373 have design stresses of 18,000 psi; A242 is used at 27,000 psi; and the new alloy steel works at 45,000 psi. This makes a fairly clean separation of types and there are steels to meet each grade. The rather large jump between the A242 level and the T-1 level indicates that there should probably be another grade somewhere around a working stress of 36,000 psi. This would provide four evenly spaced steps to work from.

Whether a fourth step is desirable is debatable. Its cost would probably be the main concern. If it begins to approach the next higher grade in cost, its use would not be economical. Being right on the margin between so-called carbon steel and alloy steel, there would probably be some pulling and hauling to see which group would make it. If it becomes an alloy steel, it would probably inherit some expensive extras to throw it out of economic competition. With steel crying for business, the criterion should be the overall approach which would manufacture the best steel at the cheapest price, regardless of who does it.

There are also many steels which fall in between grades, and there is every indication that the situation will get worse. There is no economy in furnishing a 40,000 psi steel to meet the requirements of a 27,000 psi design. Neither is it reasonable to tailor a design for a number of odd strength steels. Naturally, each steel maker would like to see the design plateaus adjusted so that his favorite steel just meets the qualifications. It will not be possible to please everybody but we certainly need some agreement on the stress classes or design plateaus so that designers will have clean stress ranges within which to choose, knowing that there will be several structural steels to meet the need.

Another need is for some clarification of the weldability of the various stress grades. A7 steel has no maximum carbon content specified. It is generally agreed that carbon content must be below 0.25% for satisfactory weldability. As a practical fact, most of the A7 steel furnished is well below the 0.25% carbon. It seems reasonable that an A7 steel could be graded by its carbon content and an A7 weldable steel offered at no premium.

Until the new A36 steel grade is definitely tied to weldability and weldability assured, it falls short of the high ideals set for it.

A242 steel, now on the second step of the stress range, is specified with a limited carbon content (0.22%) as well as restricted manganese and sulphur. However, by an unsatisfactory combination of other alloying elements, a rather unweldable steel may be obtained. Thus, we have a weldable and an unweldable steel both falling within the chemical requirements for A242. This leads to more confusion.

There have been cases, when both an A242 and a weldable A242 were specified in the same job, where suppliers rightly or wrongly jumped to the conclusion that for the ordinary A242 steel one of the common trade-named steels classed as "A242 Type" would be accepted. Without special order, these steels usually will not meet that A242 chemistry and severe disappointments have resulted.

It only increases the chaos to have steels available and pushed commercially which almost but not quite meet the accepted chemical and stress grade standards. Designers and suppliers should get together and mean the same thing when they specify certain steel grades. Also, with the increase in welding, steps should also be taken either to make all the steel furnished in a given grade completely weldable, or there should be a clear differentiation between weldable and unweldable steels in the same grade. This separating and establishing of grades must certainly be done before too long if the high-strength alloy market is not to become hopelessly tangled in its own harness.

I understand the ASTM is gradually working toward the adoption of a new steel grade in the A242 range, which will help some of this confusion.

Let us look for a minute at some of the things which show steel in a bad light in a competitive comparison.

First, of course, is the base price. It must be agreed that much of this is beyond your control. However, even this is subject to some reduction by modern methods, automation and careful planning.

Probably the second biggest factor from which you suffer in comparison is the maintenance cost. Steel corrodes and must be protected from the elements. To make this item as small as possible, you should be vitally interested in the development of the best possible protective coating for steel. Recent increased activity by the Steel Structures Painting Council is very gratifying. Until recently they were still recommending red lead and oil.

We have bad locations along the Coast where it has never been possible to make any protective coating last more than 3 or 4 years. Every 3 or 4 years we had to go back and sandblast and repaint. We tried dozens of different types of paint. We hung test panels under bridges to see how they would react. We have painted and repainted with many different materials. We have experimented with some of the new vinyl coatings and find that we now have one that has lasted eight years and will probably last ten. Thus we have found one that will last twice as long as the best one we had before. If the paint lasts twice as long, the maintenance factor is cut in half.

But with the memory of your representatives hammering at me to use more steel, I cannot help but wonder why *we* have to do all this research. Certainly we are trying to get the most for our money and we will continue to make such studies as are required to make our dollars go as far as possible. But, *you* are the ones who are trying to sell the steel.

In speaking of some of these things I am well aware that you have already launched programs of investigation along some of these lines. A year ago at your annual National Engineering Conference Mr. W. R. Jackson of the Pittsburg-Des Moines Steel Company covered the subject very well. He pointed out projects that are underway in various places in which you have some interest. It is also easy to see that he had a very real awareness of the value of these projects and their necessity in maintaining steel's position in a competitive market.

Last year also, Mr. Marvin of the American Bridge Company spoke to you on the economic design of short-span steel bridges. Mr. Marvin presented some very good ideas and pointed out the problems in short span steel design. (Parenthetically, I would disagree with his conclusion that steel is holding its own. On the west coast steel is definitely falling behind in its battle for the short span bridge.)

I can claim no originality for these ideas of what is necessary for you to do in support of your product. Your literature is full of people pointing out the need for better protective coatings, for cheaper designs, for more economical ways of fabricating steel. Is it all talk by a few, or is the industry as a whole doing something about it?

Last year it was said that your research program had cost annually between fifty and seventy thousand dollars. I still wonder if those figures aren't a misprint. This figure, I understand, is being increased. For an industry grossing as much as a billion dollars, seventy thousand dollars for research is not at all impressive. Can you think of another industry as large that spends less? I don't know what the latest figure is, but are you sure it doesn't sound like a misprint too?

For your research program you spend a modest amount on some worthwhile projects. Whether you should go further is for you to determine. It is your problem—what is it worth to you?

You are now running a test program on shear connectors for composite girders. These are the welded studs, bars, coils, etc., fastened to the top flange of a steel girder to make the concrete deck act in conjunction with it. The welding of these shear developers to the top flange is a very expensive operation. The design of these connectors is now done by a largely empirical method on very meager information. We are convinced that we are probably putting two or three times as many of these shear connectors onto our steel girders as is necessary. This unnecessarily increases the cost. However, we have no information reliable enough to permit us to make a reduction. We hope that your research will make a material reduction possible and thus reduce the final cost of these beams. This is a good practical project.

You also are participating in a series of tests aimed at determining the effectiveness of thin web plates in plate girders. This project is of great academic interest.

You are doing some very fine things. Your recent publication containing the moments, shears, and reactions for continuous highway bridges was a step in the right direction—and what made it better, it was distributed for free. Santa Claus opening his pack never had anything on the joy and fullness of heart of your representative when he came into our drafting rooms to distribute some free AISC literature.

Your seminars on plastic design also are to be commended. Your studies of orthotropic design may well lead to something good—although the problems of hand labor and corrosion resistance loom large.

I well realize many of the problems you are up against. You are an association of competing fabricators banded together for the common promotion of your industry. Nevertheless, I imagine anyone trying to raise funds for research or promotion has a rather tough time. However, promotion and research are essential to your well being. The steel industry as a whole must get together in one common purpose.

A ship was approaching shore at night. The bosun, on watch in the bow, got worried and approached the captain. "Sir, I hear breakers ahead. I suggest we change course." The captain pointed out that he was navigating the ship and that everything was under control and sent the bosun back to his post. In a couple of minutes the bosun was back again. "Sir, I distinctly hear the breakers ahead. I suggest seriously that we change course." The captain exploded: "Listen Bosun, I'm running this ship. You take care of your part of the ship and I'll take care of mine." The bosun left and a minute later there was the sound of chain running out. The bosun reported back to the bridge: "Captain Sir, my part of the ship is anchored."

The steel industry has many of the same problems. One group cannot be dropping its anchor while another tries to get ahead. The fabricators and the mills, the carbon steel people and the alloy steel people, the welding enthusiasts and those who still believe in riveting, the salesmen and the technical men—all should have a common goal, to get the most out of steel.

The new horizons for high-strength steel are clear and bright. Wherever steel is used, high-strength steel will become more and more popular to simplify details, carry greater loads, and effect more economical construction. However, these benefits also bring responsibilities. The steel industry cannot expect to partake of these fruits unless it plants and fertilizes the trees and tends and cultivates the orchard.

Discussion

(*Q*) ANONYMOUS: What type of splices did you use on the bridge?

(*A*) ELLIOTT: Butt welded splices.

* * *

(*Q*) ANONYMOUS: What design stresses are you using for T1 steel?

(*A*) ELLIOTT: 45,000 psi.

* * *

(*Q*) ANONYMOUS: Did you completely seal the box girders to prevent corrosion?

(*A*) ELLIOTT: The box is about 2½-ft. wide x 8-ft. deep. We considered sealing it completely, but decided there was too much chance of a leak somewhere. We therefore sealed it except for an inspection door, so that we could get in and have a look.

* * *

(*Q*) ANONYMOUS: What prices are used as the basis for your cost comparisons?

(*A*) ELLIOTT: We investigate the prices of bridges that are as nearly as possible like the bridge under investigation and of recent design. I think that prices used for the bridges I have described were less than three years old. We have made similar comparisons, though, over the past ten years, and the comparative results always seem to come out the same. We watch the trends in bridge design and costs to get our prices for comparison. We don't just investigate one job here and one job there.

* * *

(*Q*) ANONYMOUS: Are you comparing costs on riveted versus welded design?

(*A*) ELLIOTT: We are using only welded designs.

* * *

(*Q*) ANONYMOUS: Why do you still use only 18,000 psi stresses in your comparison?

(*A*) ELLIOTT: We follow the AASHO Code.

* * *

(*Q*) A. L. COLLIN, Kaiser Steel Corp.: Can you tell us specifically how we, as fabricators, can help your design section?

(*A*) ELLIOTT: There are a number of ways in which you can help. You claim that you can fabricate certain types of steel structures cheaper than we are estimating the cost. Show us what you can build it for, and, if you will bid on that basis, usually we would be glad to put it out as an alternate. If you quote to a contractor and he boosts your price, we'll be glad to give consideration to the true price. Things like that are just a matter of communication. It's one thing to complain and another thing to bring in something that's conservative, show where you can do the work cheaper, and give us ideas. Have you ever gone around to designers to show them some of the difficulties of fabricating? Have you ever shown them what troubles you have or how much cheaper it would be if you could do it this way or that way? We are always glad to have someone come in and show us, for example, that if a certain angle could be relocated slightly it would simplify jig work, or in some other way reduce costs. Communication with the designer can frequently iron out your problems.

* * *

(*Q*) ANONYMOUS: Do you have any hesitancy about mixing steels with different yield points? For example, in butt welding, or in welding a flange to a web, do you hesitate to use T1 with A242? What about the box section with the two side members different than the cover plates?

(*A*) ELLIOTT: Well, if you'll notice on Whiskey Creek there wasn't too great disparity of steels. We didn't have the different kinds of steels overlapping too far. However, when you start welding these different steels together, you'll find that you're controlled by deflection. This becomes more important than the stresses in some cases. The modulus of elasticity of all these steels is the same, so that deflections are quite similar despite differences in stress. You don't run into much trouble if you don't go too far in mixing the steel.

* * *

(*Q*) S. CLARK, AISC, District Engineer, San Francisco: I'd like to ask if you have any recollection of the last time you used your standard beam cover plate design for a short span bridge? I think it's been some time since a cost comparison was made.

(*A*) ELLIOTT: I don't remember how long ago, but I would guess probably about 1953.

Economic Possibilities of Corrosion—Resistant Low Alloy Steel in Short Span Bridges

J. M. HAYES* and S. P. MAGGARD

The economy of the use of high strength low alloy steels in long span highway bridges is an accepted fact. The economic possibilities of the use of these steels in short span highway bridges has not been fully studied. This is a report on a study of the economic use of nickel-copper corrosion resistant types of high strength low alloy steels in short span highway bridges which is underway at Purdue University in cooperation with The International Nickel Company, Inc.

Results of the study indicate that short span highway bridges may, for all practical purposes, be constructed of nickel-copper types of high strength low alloy steels at the same first cost in dollars as if structural carbon steel has been used. This means that the greater resistance to atmospheric corrosion of nickel-copper types of high strength low alloy steels, with the resultant increase in durability of paint coatings, may be obtained at no extra cost over the use of structural carbon steel.

Purpose of Study

The objective of the work at Purdue University is to investigate the economic possibilities of nickel-copper high strength low alloy steels in short span highway bridges. The first phase of the study has been analytical comparisons of the designs for the superstructures of typical short span concrete slab and rolled WF steel stringer highway bridges fabricated from nickel-copper high strength low alloy steels with those fabricated of structural carbon steels.

Nickel-Copper High Strength Low Alloy Steels

These steels are covered under Standard Specifications, ASTM Designation A242, which define composition in terms of maximum contents of carbon and manganese. Proprietary steels supplied to A242 Specifications contain various combinations of additional alloying elements such as nickel, copper, silicon, phosphorus, molybdenum, chromium, and vanadium to achieve the specified strength properties and to enhance atmospheric corrosion resistance in differing degrees. The economic cost analysis in this study was based on only the types within A242 Specifications that contain nickel and copper and that provide an atmospheric corrosion resistance recognized to be 4 to 6 times that of carbon steel.

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* The paper was presented by Mr. Hayes.

Details of Typical Design for Short Span Highway Bridges

A 64-foot simply supported span with a 30-foot clear roadway was considered to represent the size of a large portion of the short span highway bridges to be built in the next few years. A two-span structure, probably continuous, with each span 64 feet in length, represents about the minimum structure over the interstate highway system. In order to take into consideration possible variations in the efficiencies of the WF rolled sections available for stringers, comparative designs were made at two-foot intervals from the 64-foot span to the maximum simple span length permitted by the span-depth limitations of the American Association of State Highway Officials (AASHO), STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 7th edition, 1957—about 76 feet for non-composite design and about 94 feet for composite design.

Many details of the typical concrete slab steel stringer highway bridge are matters of personal opinion. The objective in this study was to make comparable designs in both the high strength low alloy and structural carbon steels, with the details as generally acceptable as possible. Complete superstructure designs were made for both six and seven foot interior stringer spacings. (See Fig. 1.) An even spacing of stringers probably would have served as well. Bearing and diaphragm details are as found in the Bureau of Public Roads, U. S. Department of Commerce, STANDARD PLANS FOR HIGHWAY BRIDGE SUPERSTRUCTURES, revised 1956. ASTM A7 structural carbon steel is used for the non-composite designs and ASTM A373 structural carbon steel is used for the composite designs where a plate is welded to the bottom flange of the WF rolled section.

Assumptions Made in Comparative Designs

The structures were designed for the H20-S16 live loading in accordance with the AASHO specifications

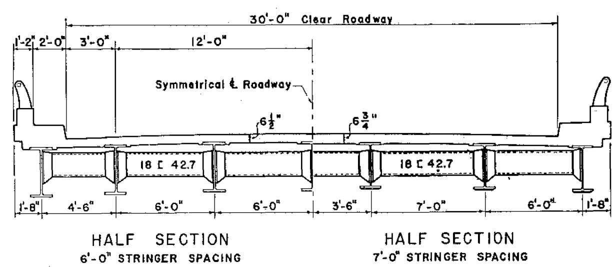

FIGURE 1
Typical Details of Comparative Designs.

all three of the concentrated loads will remain on the span when positioned for maximum bending moment. The deflection in inches under the load at point of maximum bending moment for two lanes of H20-S16 live loading and impact may then be written as

$$\left[+0.17875862 \cdot L^3 - 109.00304 \cdot L + 272.51 - 763.02 \cdot \frac{1}{L}\right] \cdot \frac{1}{I} \cdot \left[\frac{50}{L+125} + 1\right]$$

where L is the length of the simply supported span in feet and I is the sum of the moments of inertia of all the stringers in the span in inch units.

Figure 4 shows a plot of the required minimum sum of the moments of inertia of all the stringers in a simple span to maintain the specification deflection limitation of 1/800 of the span for the H20-S16 live loading and impact effect. Also shown are the required minimum sums of the moments of inertia of all the stringers in a simple span to maintain the live load plus impact deflection at both 1/1000 and 1/1200 of the span for the H20-S16 live loading. Deflection limitations are somewhat of an unsettled problem at present. Also shown are the values of the total moments of inertia furnished for both composite and non-composite designs for the 7-foot interior stringer spacing. There would be minor variations with other interior stringer spacings. This shows that the live load and impact deflection limitations of the AASHO Bridge Specifications are not critical where stringers are fabricated with WF sections rolled from A242 steels and adequate diaphragms are used.

Table 3

COMPARATIVE QUANTITIES AND COST ESTIMATES

COMPOSITE ACTION—6-FT. INTERIOR STRINGER SPACING

Steel	*Span Length ft.*	STRINGER *WF*	*Plate*	STRUCTURAL STEEL—LBS. *Str.*	*Diaph.*	*Total*	COST ESTIMATE—DOLLARS *Str.*	*Diaph.*	*Total*
A373	64	33WF130	10 x 5/8	68,390	10,300	78,690	$12,125	$1,700	$13,825
A242	64	30WF108	9 x 3/8	55,580	9,950	65,530	$12,269	$1,928	$14,197
A373	66	33WF130	10 x 3/4	71,540	10,300	81,840	$12,663	$1,700	$14,363
A242	66	30WF108	9 x 11/16	59,850	9,950	69,800	$12,987	$1,928	$14,915
A373	68	33WF130	10 x 7/8	74,830	10,300	85,130	$13,223	$1,700	$14,923
A242	68	30WF108	9 x 3/4	62,090	9,950	72,040	$13,513	$1,928	$15,441
A373	70	36WF150	11 x 9/16	84,350	10,600	94,950	$14,879	$1,750	$16,629
A242	70	30WF116	9 x 3/4	67,830	9,950	77,780	$14,975	$1,955	$16,930
A373	72	36WF150	11 x 5/8	87,360	10,600	97,960	$15,385	$1,750	$17,135
A242	72	30WF116	9 x 7/8	70,770	9,950	80,720	$15,542	$1,955	$17,497
A373	74	36WF150	11 x 3/4	91,140	10,600	101,740	$16,023	$1,750	$17,773
A242	74	33WF130	10 x 1/2	77,070	10,300	87,370	$16,524	$1,990	$18,514
A373	76	36WF150	11 x 7/8	94,990	10,600	105,590	$16,671	$1,750	$18,421
A242	76	33WF130	10 x 5/8	80,430	10,300	90,730	$17,210	$1,990	$19,200
A373	78	36WF160	11 x 7/8	102,900	10,600	113,500	$17,905	$1,751	$19,656
A242	78	33WF130	10 x 11/16	83,090	10,300	93,390	$18,098	$1,990	$20,088
A373	80	36WF160	11 x 1	106,960	10,600	117,560	$18,461	$1,753	$20,214
A242	80	33WF130	10 x 13/16	86,520	10,300	96,820	$18,716	$1,990	$20,706
A373	82	36WF170	11 x 1	115,500	12,720	128,220	$19,762	$2,106	$21,868
A242	82	33WF141	10 x 3/4	94,360	12,360	106,720	$20,178	$2,388	$22,566
A373	84	36WF170	11 x 1 1/8	119,840	12,720	132,560	$20,337	$2,107	$22,444
A242	84	33WF141	10 x 7/8	98,070	12,360	110,430	$20,830	$2,388	$23,218
A373	86	36WF182	11 x 1 3/16	130,690	12,720	143,410	$21,982	$2,110	$24,092
A242	86	36WF150	11 x 5/8	103,600	12,720	116,320	$22,210	$2,424	$24,634
A373	88	36WF194	11 x 1 3/16	141,120	12,720	153,840	$23,525	$2,112	$25,637
A242	88	36WF150	11 x 3/4	107,660	12,720	120,380	$23,370	$2,424	$25,794
A373	90	36WF230	15 x 5/8	162,890	12,600	175,490	$26,909	$2,096	$29,005
A242	90	36WF150	11 x 13/16	110,880	12,720	123,600	$25,380	$2,420	$27,800
A373	92	36WF230	15 x 11/16	167,580	12,600	180,180	$27,684	$2,096	$29,780
A242	92	36WF150	11 x 15/16	115,080	12,720	127,800	$26,219	$2,420	$28,639
A373	94	36WF230	15 x 13/16	173,600	12,600	186,200	$28,679	$2,096	$30,775
A242	94	36WF160	11 x 15/16	124,110	12,720	136,830	$27,686	$2,420	$30,106

Figure 5 shows a plot of the actual H20-S16 live load and impact deflections in inches for interior stringers with 7-foot spacing where the deflections are computed for the same load distribution as was used in computing the critical extreme fiber stresses. It should be noted that non-composite designs of A7 steel do not satisfy the specification deflection requirement under these assumptions of load distribution. It is seen that only the use of A373 steel in composite action will satisfy the specification deflection requirements under these assumptions of load distribution. Even under these assumptions of load distribution, the deflection problem should be less critical with the use of built-up stringers. The next phase of the investigation will include a study of built-up stringers. References to load distribution in the I-beam bridge may be found in the bibliography at the end of the paper.

Comparative Quantities

Although the 64-foot simple span was considered to represent the size of a large portion of the short span highway bridges to be built in the next few years, comparative quantities were computed for simple spans at two-foot intervals from 64 to 76 feet for non-composite action and from 64 to 94 feet for composite action.

These comparative quantities do not include the shear connectors, since it was assumed that the quantities and type involved would be about the same regardless of the steel used. Another variable is differences in concrete slab quantities for changes in stringer spacing. This

Table 4

Comparative Quantities and Cost Estimates
Composite Action—7-Ft. Interior Stringer Spacing

Steel	Span Length ft.	Stringer WF	Stringer Plate	Structural Steel—Lbs. Str.	Structural Steel—Lbs. Diaph.	Structural Steel—Lbs. Total	Cost Estimate—Dollars Str.	Cost Estimate—Dollars Diaph.	Cost Estimate—Dollars Total
A373	64	33WF141	10 x 7/8	64,860	9,750	74,610	$11,266	$1,609	$12,875
A242	64	30WF108	9 x 5/8	49,380	9,500	58,880	$11,072	$1,833	$12,905
A373	66	36WF150	11 x 11/16	69,540	10,000	79,540	$12,016	$1,651	$13,667
A242	66	30WF116	9 x 7/8	55,920	9,500	64,420	$12,123	$1,856	$13,979
A373	68	36WF150	11 x 13/16	72,660	10,000	82,660	$12,491	$1,651	$14,142
A242	68	33WF130	10 x 1/2	60,960	9,750	70,710	$12,872	$1,881	$14,753
A373	70	36WF150	11 x 15/16	75,900	10,000	85,900	$12,979	$1,651	$14,630
A242	70	33WF130	10 x 5/8	63,780	9,750	73,530	$13,393	$1,881	$15,274
A373	72	36WF160	11 x 15/16	82,320	10,000	92,320	$14,003	$1,651	$15,654
A242	72	33WF130	10 x 3/4	66,600	9,750	76,350	$14,193	$1,881	$16,074
A373	74	36WF170	11 x 15/16	88,980	10,000	98,980	$15,064	$1,651	$16,715
A242	74	33WF130	10 x 7/8	69,480	9,750	79,230	$15,018	$1,881	$16,899
A373	76	36WF170	11 x 1 1/16	92,520	10,000	102,520	$15,580	$1,651	$17,231
A242	76	33WF141	10 x 13/16	75,720	9,750	85,470	$15,914	$1,881	$17,795
A373	78	36WF182	11 x 1 1/8	101,220	10,000	111,220	$16,904	$1,652	$18,556
A242	78	33WF141	10 x 15/16	78,840	9,750	88,590	$16,856	$1,881	$18,737
A373	80	36WF194	11 x 1 1/8	109,560	10,000	119,560	$18,155	$1,653	$19,808
A242	80	36WF150	11 x 11/16	83,460	10,000	93,460	$17,905	$1,906	$19,811
A373	82	36WF194	11 x 1 1/4	113,700	12,000	125,700	$18,681	$1,985	$20,666
A242	82	36WF150	11 x 13/16	86,940	12,000	98,940	$18,468	$2,287	$20,755
A373	84	36WF230	15 x 11/16	131,580	11,880	143,460	$21,448	$1,966	$23,414
A242	84	36WF150	11 x 15/16	90,420	12,000	102,420	$20,114	$2,276	$22,390
A373	86	36WF230	15 x 13/16	136,560	11,880	148,440	$22,068	$1,967	$24,035
A242	86	36WF160	11 x 15/16	97,680	12,000	109,680	$21,103	$2,276	$23,379
A373	88	36WF230	15 x 7/8	140,640	11,880	152,520	$22,545	$1,970	$24,515
A242	88	36WF160	11 x 1	100,560	12,000	112,560	$21,547	$2,278	$23,825
A373	90	36WF230	15 x 1	145,800	11,880	157,680	$23,168	$1,970	$25,138
A242	90	36WF170	11 x 1 1/16	108,960	12,000	120,960	$22,645	$2,278	$24,923
A373	92	36WF230	15 x 1 1/8	151,080	11,880	162,960	$24,007	$1,970	$25,977
A242	92	36WF182	11 x 1	117,240	12,000	129,240	$23,924	$2,278	$26,202
A373	94	36WF245	15 x 1 1/8	162,780	11,880	174,660	$25,865	$1,970	$27,835
A242	94	36WF182	11 x 1 1/8	121,260	12,000	133,260	$25,241	$2,278	$27,519

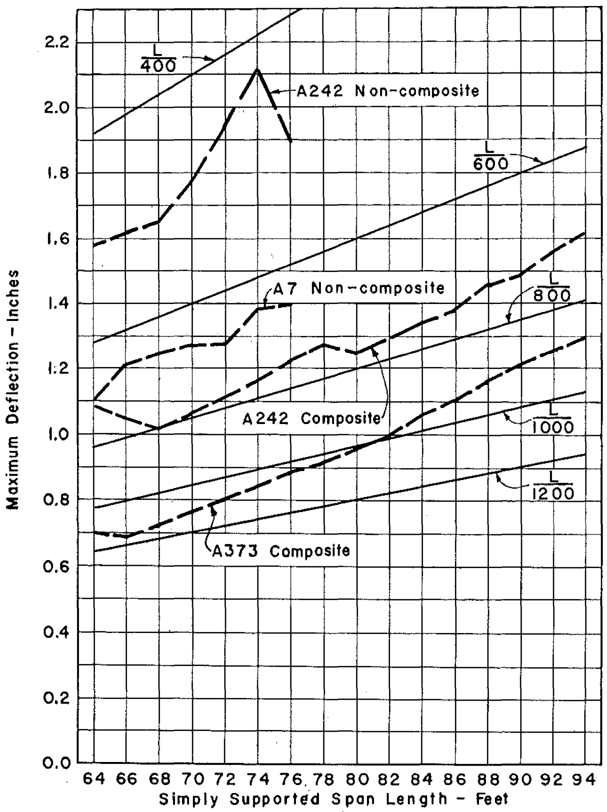

FIGURE 5
Live Load Plus Impact Deflection Limitations Based on Design Stress in Interior Stringer. 7 ft. Stringer Spacing.

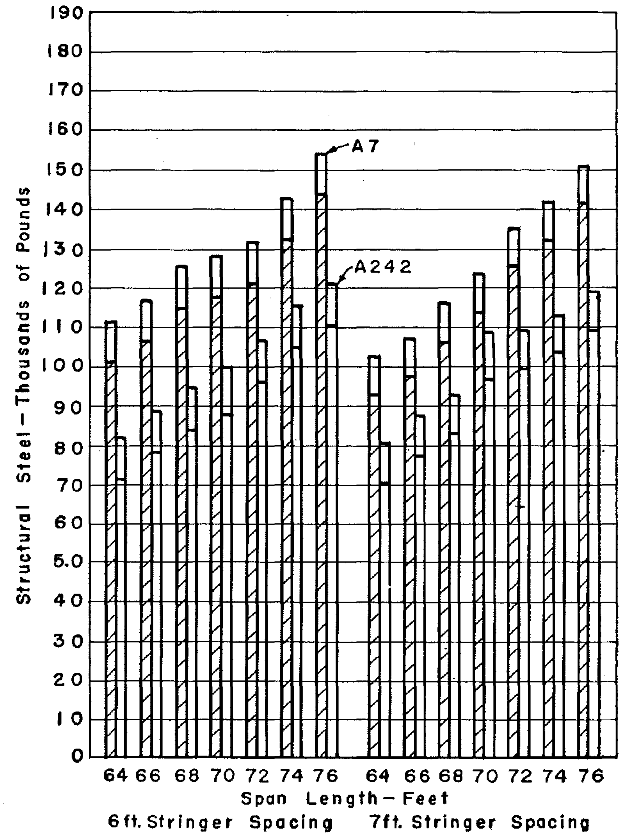

FIGURE 6
Structural Steel—Non-composite Action.

has been neglected. Still another variable is a difference in depth from grade to low steel. This might mean a reduction in quantities in overpass structures due to a decrease in height of the approach fill. No substructure items are considered in these comparisons. The use of high strength low alloy steels would probably not reflect any savings in the substructure quantities for short span highway bridges as compared to the use of A7 or A373 steels. This may be an important item as the span length of the bridge increases. In practice, each structure must be evaluated on its own merits.

The structural steel quantities are designated in Tables 1, 2, 3, and 4 as stringer or diaphragm quantities. The stringers include the main beam, bottom plate, bearing details, and bolts for field connections. All diaphragms are 18-inch, 42.7 pound channels using split T connections with a maximum spacing of 20 feet. In all instances the diaphragms and connections are of the same type of steel as the stringers.

A comparison of the structural steel quantities is shown graphically in Figures 6, 7, 8 and for the various span lengths and stringer spacings for both composite and non-composite action. The stringer quantities are plotted from the origin with the diaphragm quantities plotted directly on top of the stringer quantities. The A7 or A373 steel quantities are on the left of each span comparison with the portion of the bar diagram representing the stringer steel hatched.

Comparative Cost Estimates

Comparative cost estimates in dollars are shown in Tables 1, 2, 3, and 4. Detailed cost analyses were made for the 64 and 76 foot spans for non-composite action and for the 64, 76 and 90 foot spans for composite action —for both the six and seven foot interior stringer spacings. The structural steel was grouped into two divisions —stringers and diaphragms. A separate unit cost analysis was made for each grouping. Each piece of material was individually priced from the mill base with appropriate charges for all extras, including camber. Freight from mill to fabricating plant to distribution point was assumed at an average of one cent per pound. Costs in dollars of engineering, fabricating, erecting, and painting were assumed the same regardless of the type of steel used. The slightly higher cost of fabricating A242 steel is practically offset by the lesser weights to be handled in the shop and field. A unit cost analysis of these items was made for A7 or A373 steels and the same total cost was used for the A242 steel.

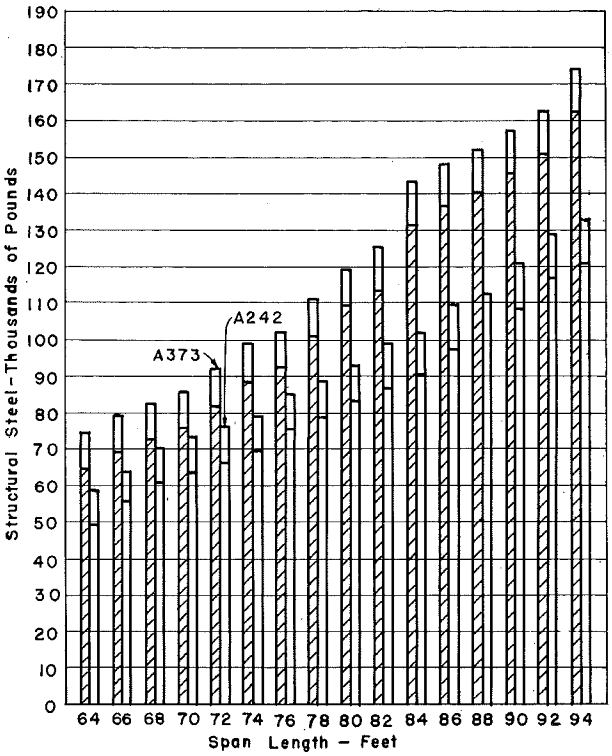

FIGURE 7

Structural Steel—Composite Action. 7 ft. Stringer Spacing.

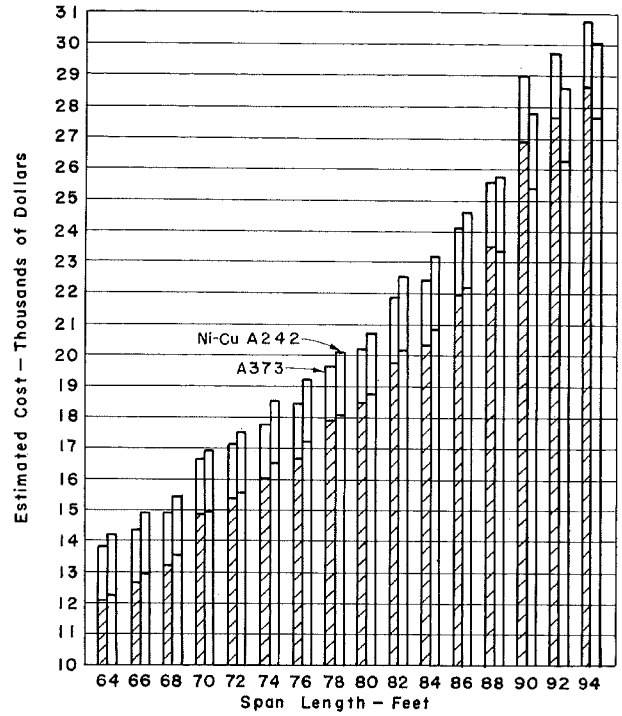

FIGURE 8

Structural Steel—Composite Action. 6 ft. Stringer Spacing.

Summaries of the cost analyses are given in Tables 5, 6, and 7 for the 64, 76, and 90 foot spans respectively. Table 7 shows the analysis for the diaphragm steel for the 90 foot span. Similar analyses were made for the diaphragm steel in the 64 and 76 foot spans, which gave practically the same unit costs.

A comparison of the total cost estimates is shown graphically in Figures 9, 10, and 11 for the various span lengths and stringer spacings for both composite and non-composite action. The stringer costs in dollars are plotted from an origin of $10,000 with the diaphragm costs in dollars plotted directly on top of the stringer costs. The cost estimates for the A7 or A373 steel are on the left of each span comparison with the portion of the bar diagram representing the stringer cost estimate hatched.

Table 5

SUMMARY OF COST ANALYSIS
UNIT PRICE—CENTS PER POUND
STRUCTURAL STEEL—STRINGERS
64-FOOT SIMPLE SPAN

	6-Ft. Stringer Spacing				7-Ft. Stringer Spacing			
	Composite		*Non-Composite*		*Composite*		*Non-Composite*	
	A242	*A373*	*A242*	*A7*	*A242*	*A373*	*A242*	*A7*
Material base, extras and camber	9.41	7.25	9.38	6.58	9.43	7.24	9.17	6.52
Freight	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Engineering, fabricating, erecting and painting	*	9.48	*	6.79	*	9.13	*	6.60
Total Price		17.73		14.37		17.37		14.12

* Same total cost for these items as A7 or A373

Table 6

SUMMARY OF COST ANALYSIS
UNIT PRICE—CENTS PER POUND
STRUCTURAL STEEL—STRINGERS
76-FOOT SIMPLE SPAN

	6-Ft. Stringer Spacing				7-Ft. Stringer Spacing			
	Composite		*Non-Composite*		*Composite*		*Non-Composite*	
	A242	*A373*	*A242*	*A7*	*A242*	*A373*	*A242*	*A7*
Material base, extras and camber	9.59	7.40	9.44	6.70	9.57	7.29	9.21	6.68
Freight	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Engineering, fabricating, erecting and painting	*	9.15	*	6.32	*	8.55	*	5.87
Total Price		17.55		14.02		16.84		13.55

* Same total cost for these items as A7 or A373

Table 7

SUMMARY OF COST ANALYSIS
UNIT PRICE—CENTS PER POUND
STRUCTURAL STEEL—STRINGERS AND DIAPHRAGMS
90-FOOT SIMPLE SPAN
COMPOSITE ACTION

	6-Ft. Stringer Spacing				7-Ft. Stringer Spacing			
	Stringers		*Diaphragms*		*Stringers*		*Diaphragms*	
	A242	*A373*	*A242*	*A373*	*A242*	*A373*	*A242*	*A373*
Material base, extras and camber	9.74	7.25	9.04	6.56	9.56	7.25	9.00	6.51
Freight	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Engineering, fabricating, erecting and painting	*	8.27	*	9.07	*	7.64	*	9.07
Total Price		16.52		16.63		15.89		16.58

* Same total cost for these items as A373

The total estimated cost using a nickel-copper grade of A242 steel for the six-foot interior stringer spacing with non-composite action varies from 0.3 percent to 5.4 percent greater than the cost using A7 steel. The variation is from 2.6 percent to 8.3 percent greater for the seven-foot interior stringer spacing.

The total estimated cost using a nickel-copper grade of A242 steel for the six-foot interior stringer spacing with composite action varies from 4.5 percent greater to 4.2 percent less than the cost using A373 steel. The variation is from 3.2 percent greater to 2.8 percent less for the seven-foot interior stringer spacing.

These variations reflect the comparative efficiency with which the rolled WF section and plates may be used for a specific span length. In general, it might be stated that the cost is about the same regardless of the steel used. Each design should receive individual attention.

This economic evaluation does not include possible long-term maintenance savings due to the increased resistance to atmospheric corrosion and the better paint life of nickel-copper types of high strength low alloy steels. This will require individual study for each structure or like grouping of structures in the same locality, but general information that has been accumulated on this subject can be cited.

High Strength Low Alloy Steel Paint Life

A great deal of work on the durability of paint coatings on steel by many investigators has confirmed the belief that any improvement in the corrosion resistance of the steel produces a beneficial effect on the durability of paint coatings. These benefits include longer life for the paint, less pitting of the steel, and a better surface for repainting. Investigations made under a great variety of

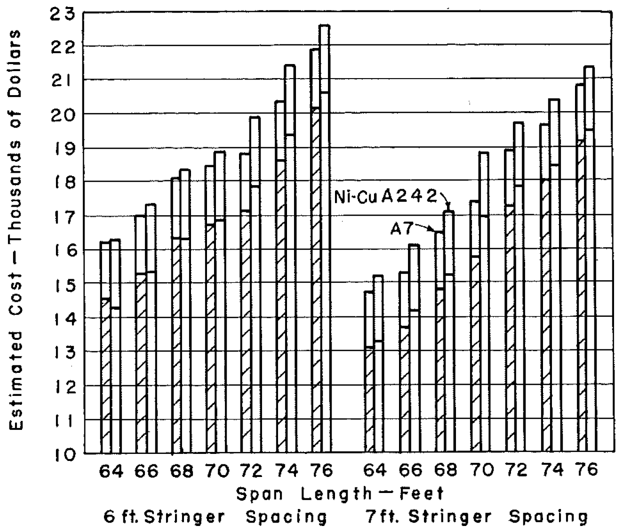

FIGURE 9
Estimated Cost—Non-composite Action.

atmospheric exposure conditions indicate that the advantages displayed by certain alloy steels can be traced to the nature and amount of rust that forms on them.

Among the combinations of alloying elements that may be used to achieve the strength properties of A242 steels, those incorporating nickel and copper, frequently in combination with other alloying elements, produce the thinner, less permeable, more tenacious rust coatings that are more protective than those of carbon steels. Evidence on

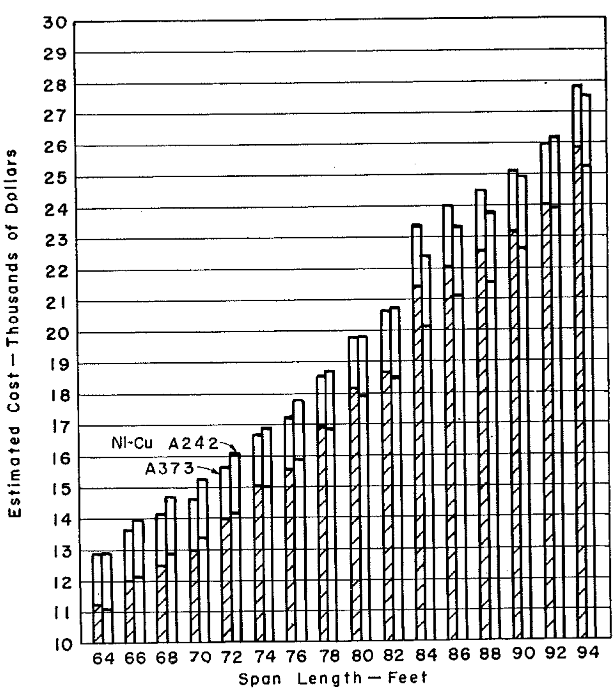

FIGURE 10
Estimated Cost—Composite Action. 7 ft. Stringer Spacing.

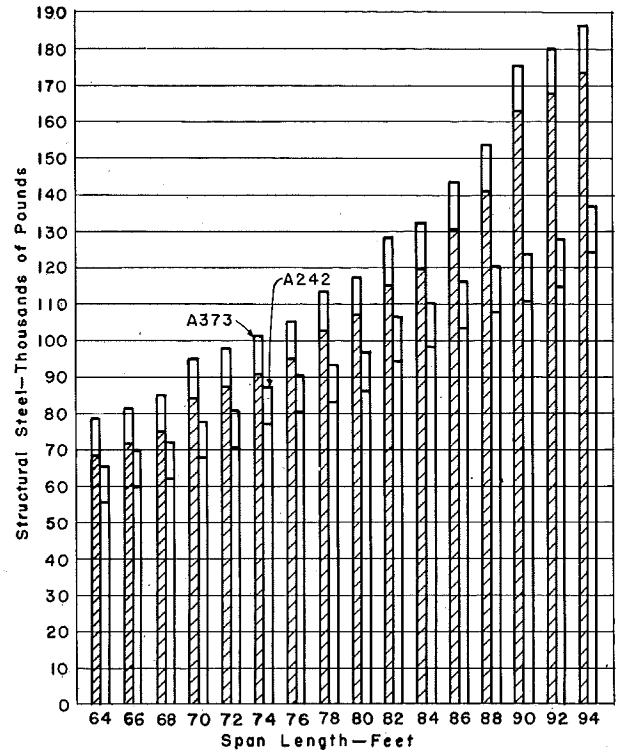

FIGURE 11
Estimated Cost—Composite Action. 6 ft. Stringer Spacing.

this point can be found in the work of LaQue, Boylan, Copson, Larrabee and a number of others cited in the bibliography. Their findings can be summarized as follows: (1) Rates of deterioration of the low alloy steels fall into two general orders: first, that typified by the steels with atmospheric corrosion resistance equal to that of copper steel; i.e., twice the corrosion resistance of carbon steel, and second, 4 to 6 times the corrosion resistance of carbon steel which is typical of the higher copper nickel types. (2) In the unpainted condition the nickel-copper types deteriorate more slowly in both marine and industrial atmospheres. (3) These advantages, due to improved corrosion resistance, persist in the painted condition. (4) The nickel-copper types provide at least 50 percent longer paint life than does carbon steel.

Conclusions

It is seen that short span highway bridges may be fabricated and erected using a nickel-copper grade of high strength low alloy steel in accordance with the AASHO Bridge Specifications at practically the same cost as if A7 or A373 carbon steels were used, if adequate diaphragms are used in order that full live load and impact deflection may be equally distributed to all stringers. Therefore, it seems reasonable to suggest that good engineering practice would dictate the giving of individual attention to each bridge site, or to like groupings of structures in the same locality, with respect to the possible use of nickel-copper high strength low alloy steels, since any savings in maintenance costs due to the use of these steels may be obtained at practically no additional cost over the use of carbon steels.

Acknowledgments

This study was made in the Engineering Experiment Station, Purdue University, Lafayette, Indiana, in cooperation with The International Nickel Company, Inc., New York, New York. The cost analyses, summaries of which are shown in Tables 5, 6, and 7, were made by Mr. Joseph O. May, Consulting Engineer, Ridgewood, New Jersey. The authors wish to especially thank Mr. L. Anselmini of The International Nickel Company, Inc., for his assistance in making the study and in the preparing of this report.

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Further Tests on Welded Plate Girders

KONRAD BASLER

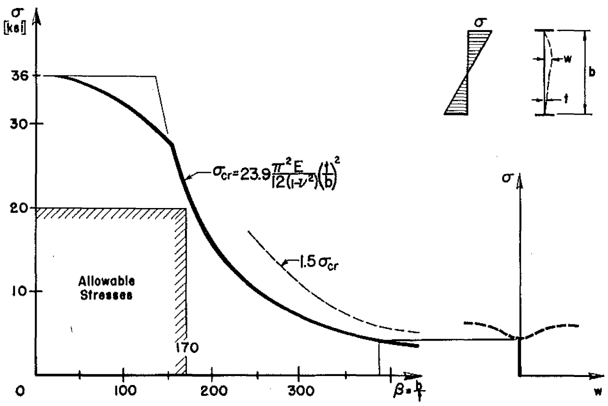

FIGURE 1

The main objective of the research project carried out at Lehigh University has been explained thoroughly at the 1958 National Engineering Conference (Ref. 1). With the help of Figure 1 it shall be reviewed briefly. Plotted in this graph is the anticipated web buckling stress, σ_{cr}, versus the ratio of web depth to web thickness. Web buckling would mean that the straight web, shown in the upper right corner of Figure 1, suddenly deflects laterally; or, using the diagram to the right, the web deflections would be zero up to the critical load and then suddenly increase. The presently used specifications for plate girders are based on this web buckling concept. It is seen that, with an allowable stress of 20 ksi, the limit of about 170 for the web depth over web thickness ratio should not be exceeded. It can be realized further why for a high strength steel with correspondingly higher allowable stresses this web slenderness limit is reduced to even lower values.

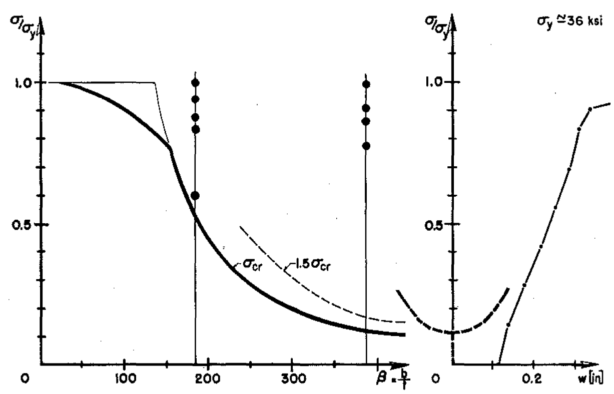

FIGURE 2

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Figure 2 repeats the discussed diagram again but it contains the test results as obtained on the girders subjected to bending. It is seen that there is little drop in the ultimate bending stress when the web slenderness limitations of today's specifications are exceeded. And how does the web behave? The unavoidable initial web deflections due to fabrication increase gradually with the applied load, not showing any sudden change when exceeding the predicted web buckling limit. As seen, the main objective of this investigation was the problem of the web stability of plate girders. The significance of the conducted tests is that they convincingly demonstrate that the web buckling theory is unable to predict the carrying capacity of plate girders and that, therefore, the specifications for plate girders should not be based on this theory.

To this experimental investigation was added a theoretical study which answers the question: If not web buckling, what then does determine the strength of plate girders and how can their strength be predicted? Based on these two investigations new design recommendations will be drafted. Figure 3 reflects this thinking and indicates, at the same time, the adopted scheme of publication. At present, the report on the conducted experiments is completed and ready for publication in the Bulletin Series of the Welding Research Council. It is contemplated to follow with the ultimate load theory in the Proceedings of the ASCE. As seen, each of the two groups are subdivided, investigating the bending strength, the shear strength, and the interaction between bending and shear.

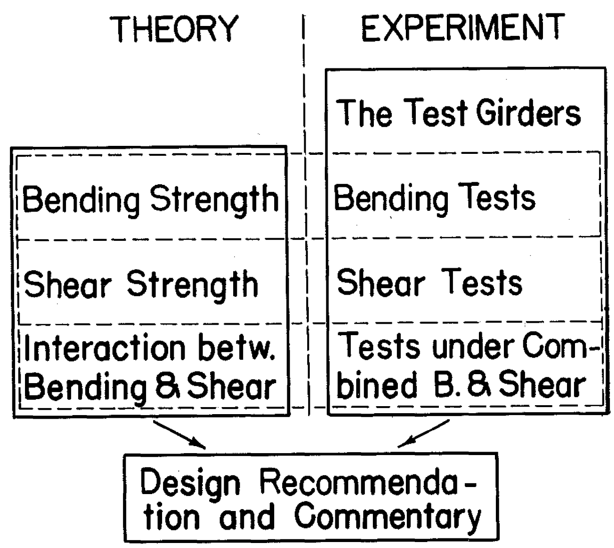

FIGURE 3

In order to illustrate the type of results emanating from this theoretical study, I intend to go briefly through the first two topics, bending strength and shear strength. Thereafter, in the last portion of this presentation, some commentary to this test report shall be given.

How will the web slenderness ratio affect the bending strength of plate girders? If the usual section modulus concept is used for the design, then the nominally computed ultimate bending stress is somewhat reduced at high web slenderness ratios because the web participation in carrying the bending moment is less than what the ordinary beam theory takes into account. For low values of the web slenderness ratio the web participation is better, approaching a stress distribution of a fully plastified web. This is pictured in Figure 4* by using the same diagram as introduced in Figures 1 and 2, correlating the extreme fiber stress with the web slenderness ratio of a plate girder. By applying a certain factor of safety the allowable bending stress appears as a function of the web slenderness ratio. This allowable stress is not yet settled; therefore, new design specifications are not available now. But it can be seen that it will be possible to build plate girders in the web slenderness range beyond 170 without introducing a longitudinal stiffener.

Having discussed briefly the girder's bending strength we shall next consider the shear strength of a transversely stiffened plate girder. Plotted as ordinate in

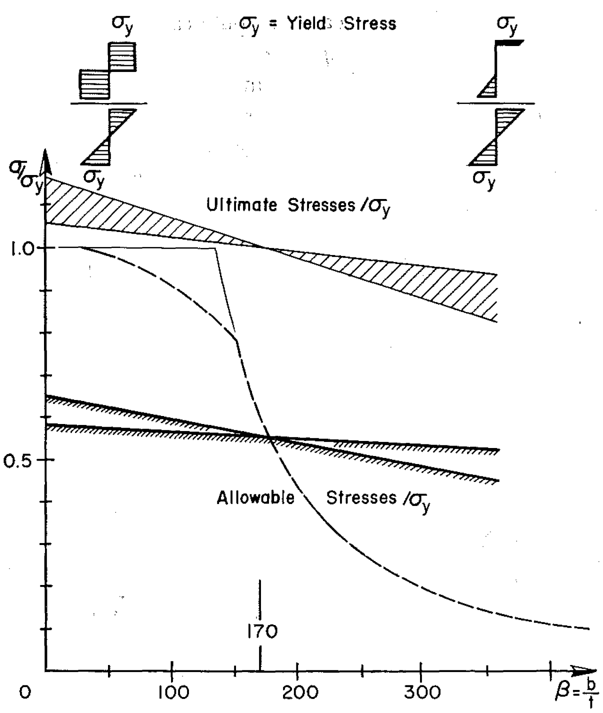

FIGURE 4

* The derivation and detailed description of these figures appears in Reference 2.

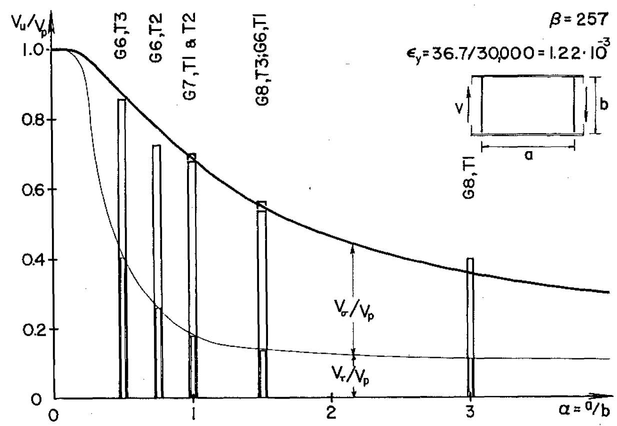

FIGURE 5

Figure 5* is the shear force which can be sustained by a girder with a web slenderness ratio of 257. (The non-dimensionalizing factor V_p is the product of shear yield stress and the web area). The numbers along the abscissa give the stiffener spacing measured in terms of the web depth. The thin line gives the prediction of the web buckling theory, while the heavily drawn line is the strength predicted by the new ultimate load theory and the height of the columns represent the observed ultimate shear force on test girders. The graph shows that these girders exhibit a shear strength which can be as much as three times the value predicted by the web buckling theory. This finding should result either in a saving in web area or an increase in the transverse stiffener spacing, because even with wider spacing the strength predicted by the web buckling theory can be obtained.

This "post buckling strength" is due to a way of carrying the shear which, so far, has not been taken fully into account by the civil engineering profession. It is due to a tension field or truss-type action. Figure 6 is a photograph of a girder failed in shear, indicating such an action. The web slenderness ratio of this girder is 130, the stiffener distance 1.5 times the web depth, and the girder was built of A7 steel.

After considering some of the results emanating from this theoretical investigation we shall next turn our at-

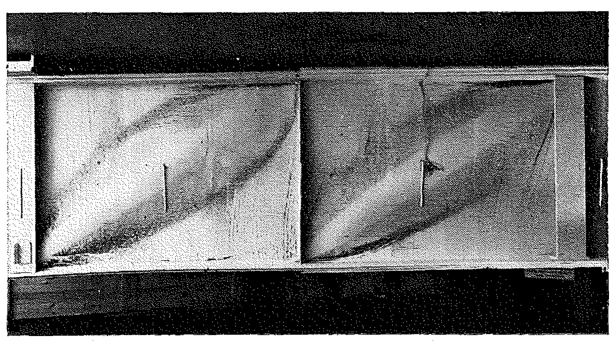

FIGURE 6

tention to the test report (Ref. 3). The test report contains information which may be of direct interest to the fabricator.

Among many other graphs, the test report includes a number of observations as shown in Figure 7 where the web deflections of the different girders are recorded. The upper portion of this figure indicates the elevation of the girder's test section and, at their respective cross sections, the web distortions to a scale twelve times that of the girder. Two states of web deflections are given, those which were there initially, i.e., were produced during fabrication, and the ones observed at about 90% of the girder's ultimate load. The left half of this test section, which is subdivided by transverse stiffeners, has fairly small initial deflections. On the right side the welding distortions are about three times bigger. When this girder was tested, failure occurred in this right hand panel. It was reinforced and a second, independent test produced failure on the left hand side. The load versus centerline deflection diagram (Fig. 2.8 in Ref. 3), records little difference in ultimate load. The conclusion is that the larger initial web deflection on the right hand side of the test section did not impair the strength. Indeed, failure in the first test occurred by lateral buckling of the compression flange which is unaffected by the web distortions.

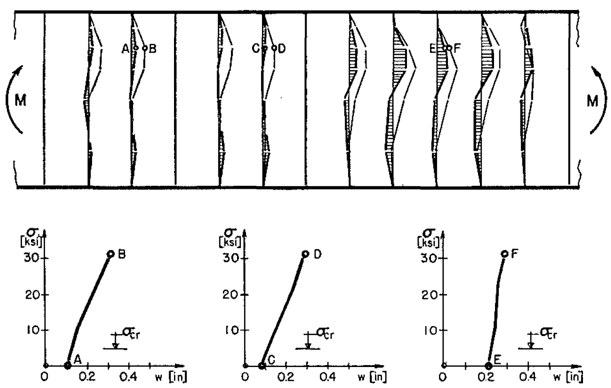

FIGURE 7

Considering the change in web deflections throughout the course of loading, it is observed in the lower half of Figure 7 that near ultimate load the deflections were all of about the same magnitude despite different initial deflections. In panels with small initial web deflections the rate of web deflection increments are generally bigger than in panels with already sizable initial distortions.

Of course, attention for proper alignment of the web plate with respect to the flange center should be given, and the welding sequence chosen such that welding distortions result as small as possible. But there will always be an unavoidable set of web deflections in a welded plate girder. It is recommended that they be left as is. One should not try with spot heating to straighten the web because this cannot improve the performance or strength of the plate girder, but only introduce more residual stresses and embrittle local web areas.

The reason why initial web deflections have little effect on the girder's strength is due to the fact that the ultimate bending strength is determined by the flanges rather than the web. And, in the case that shear governs the failure, it is obvious that the development of a tension field in the web is not dependent on the stipulation that the web be perfectly plane. This latter supposition, however, is an essential assumption for the computation of web buckling stresses. From this interpretation it can also be concluded that a web of higher strength steel does not require smaller initial deflections than that built of mild steel.

Another item which is not generally known is how to conduct a test on a welded structure and to learn what observations to make. If, for instance, a welded bridge is to be tested then one should not, while carefully loading it for the first time, read gages and try to compare them with the theoretical computation. This leads mostly to disappointment, for the first loading cycle is, in a welded structure, very misleading. As an example Figure 8 is given, showing a typical load versus the centerline

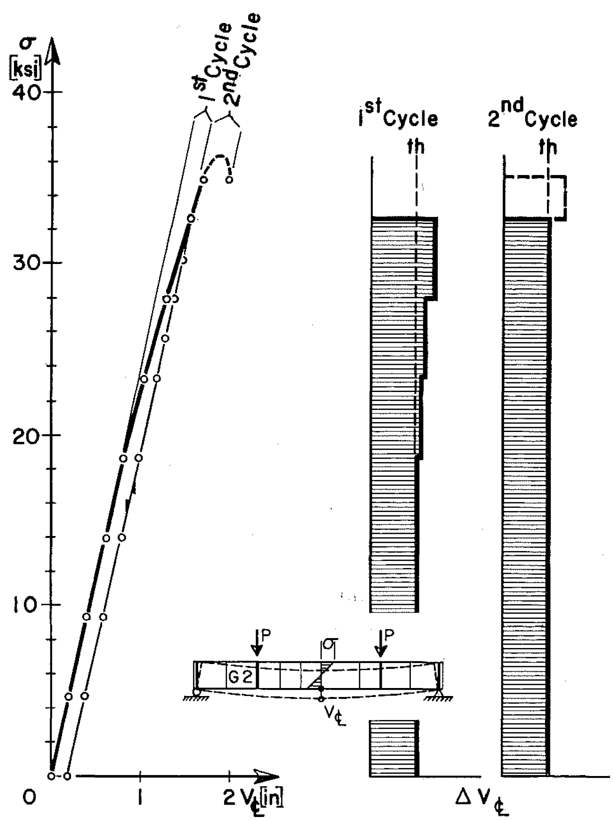

FIGURE 8

deflection of a welded plate girder (part of Fig. 2.5 in Ref. 3). Note how the deflection increments, shown on the right of the graph, increases with higher load, causing a clear deviation from the predicted, straight lined deformation and resulting in a permanent set of deflection. But in the second loading cycle, conducted within the previous load range, the correlation is perfect. This curious behavior is due to residual stresses which are partially eliminated in the first loading cycle. The following model shall explain it.

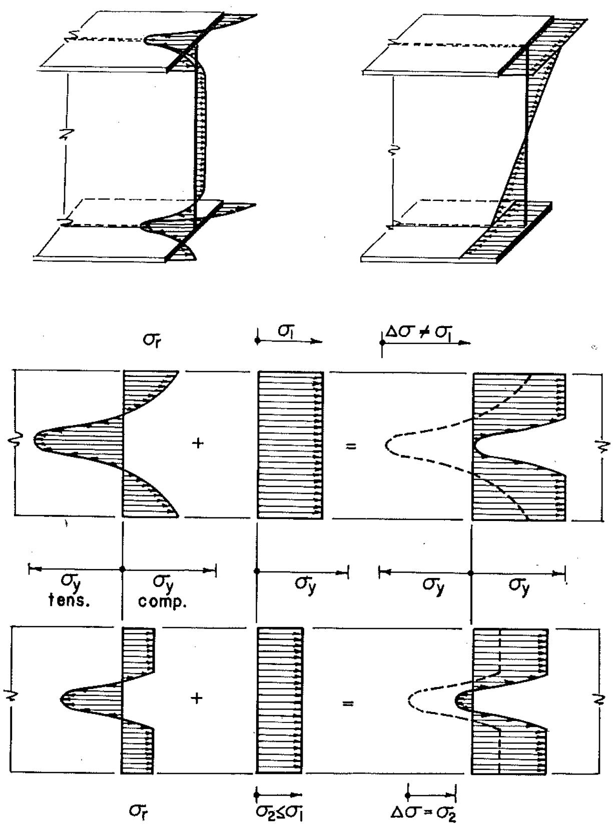

FIGURE 9

A likely state of residual stresses is shown in Figure 9 and the stresses due to applied bending moment shall be added to it. In superimposing these two stress distributions, the compression flange stresses are as shown in the second row of this figure. It is obvious that the change of stress, $\Delta\sigma$, is not uniform across the plate. The yield stress σ_y is first reached along the flange tips. Due to redistribution of stresses to fulfill the equilibrium requirements, the stress at the center line of the plate is not simply $\sigma_r + \sigma_l$. That is, if a strain gage is mounted at this point, the strain recorded by the difference of gage readings will not equal the predicted value σ_l. After reducing the applied load, the residual stresses take a new pattern with lower magnitudes as depicted in the last row of Figure 9. For subsequent loadings, within the magnitude of the previous cycle, no further yielding occurs. Thus, the measured and the computed stresses do agree with each other.

The conclusions are that not only in research but also in field tests on welded structures the first load application should not be used to judge the performance of the structure on the correlation with the theoretical predictions. A proof test, therefore, should include at least two loading cycles, and it is encouraged to go with the first as far as one can reasonably go without causing damage. This serves as a partial stress releaving.

By studying the various girder tests one realizes that the ultimate load is not the only measure of safety. Steel has the excellent ability to undergo yielding and then to strain harden. When a girder exhibits such a "deformation capacity" as indicated from the girders G3 and G5 with their load versus centerline deflection in Figure 10, then ample warning would be obtained before failure occurs. Furthermore, in a continuous girder, a moment

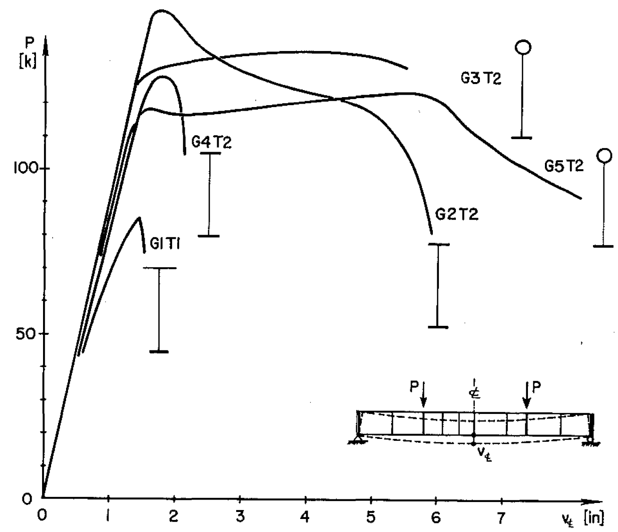

FIGURE 10

distribution could take place. With symmetrically proportioned cross sections such a favorable load deflection characteristic is hard to achieve. The compression flange, which buckles like a column, needs to be braced so closely that it can strain harden. However, if unsymmetrical girders are built such that tension flange yielding occurs just prior to compression flange instability, the desired rotation capacity can be expected.

Certainly it is, in such a case, preferable to have no plate splice at a point of maximum bending moment. Unavoidably, this happened to be the case in the last series of girders tested. Figure 11 shows a detail of a yielded flange splice under tension. In no case occurred a weld fracture which would affect the ultimate load of a girder. Again, to proof test welds was not the objective of the investigation. But the excellent performance of all the welds in the total of 13 plate girders on which more than 30 ultimate load tests were conducted, bears mute testimony as to the reliability of present welding procedures.

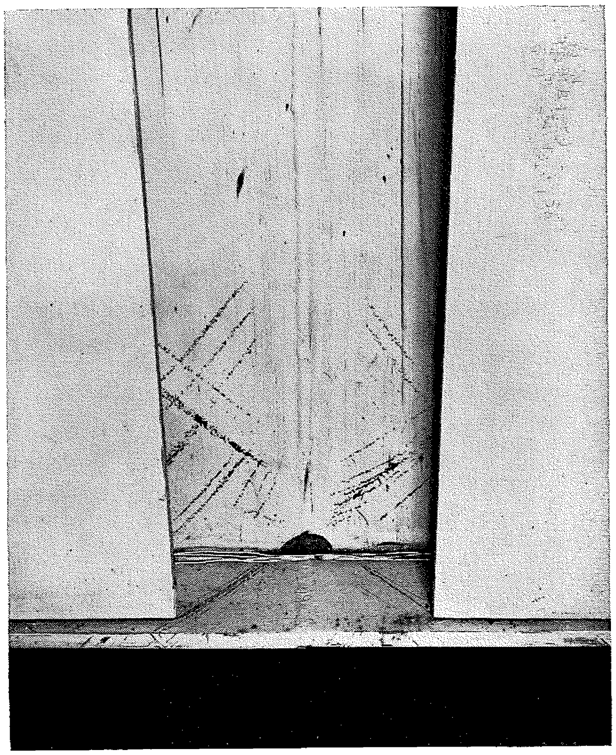

FIGURE 11

As you can see, one cannot conduct research on large size, built-up members without encountering detail problems such as I have mentioned, although they have nothing to do with the actual objective of the research. We would like to see these explanations added to one treated at last year's Engineering Conference, the detailing of transverse stiffeners (Ref. 4). The additional tests confirm again the fact that cutting transverse stiffeners short at the tension flange side does not impair the ultimate load of the girders.

The thoughts presented here are only a selection of ideas to which the investigators were inspired while conducting the tests. We hope that the study of the test reports enables the designer to obtain a feeling for the behavior of plate girders which goes beyond what dry specification rules will tell.

Acknowledgment

The Plate Girder Research Project is being carried out at Fritz Engineering Laboratory, Lehigh University, Bethlehem, Pennsylvania. W. J. Eney is Director of the Laboratory and L. S. Beedle the Chairman of the Structural Metals Division. Project Director was Bruno Thürlimann until he left for Switzerland, where he is now Professor of Structural Engineering at the Federal Institute of Technology. Bung-Tseng Yen and John A. Mueller, both former graduate students, were exclusively engaged in this Research Project. Credit belongs to everyone mentioned for his share of support and effort contributed to this investigation.

The Welded Plate Girder Project is jointly sponsored by the American Institute of Steel Construction, the Pennsylvania Department of Highways, the U. S. Department of Commerce Bureau of Public Roads, and the Welding Research Council. It is supervised by the Welded Plate Girder Project Committee. The financial support of the sponsors and the continued interest and guidance which the members of the committee have given to the project is gratefully acknowledged.

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FRIDAY AFTERNOON SESSION

May 6, 1960

Presiding: PAUL J. FOEHL, Chief Engineer, Midwest Steel and Iron Works Co., Denver, Colo.

Simple Space Structures in Steel

MILO S. KETCHUM

Increased strength and performance of engineering structures may be obtained by two different basic approaches. The first is by increasing the strength and performance of the material of which the structure is made. The second is by utilization of new shapes and forms. An example of the first approach is the use of plastic design of steel structures. The properties of the steel are utilized to obtain a much more rational design for ultimate load and thereby achieve a greater economy of material than is possible with the previous concept of the structural behavior of steel.

The second approach has for its object the shaping of the structure so that the stresses are smaller and the amount of material is reduced. An example is the rigid frame building with tapered and haunched members. It is obvious that there is less material but the cost of fabrication is increased. Our experience in this field is that the tapered frame will usually cost less than the frame with straight members. However, we have little experience to compare plastic design with the use of tapered members to reduce the amount of steel.

There is another fertile field for obtaining efficiency through shape: the space structure. An absolutely precise definition of a space structure is difficult to formulate, but in general they include those structures requiring consideration of forces in more than two directions for individual components or a series of components. A characteristic of space structures is that forces often take a shorter path than for two dimensional structures and heavy moment connections are avoided. Most of the elements work as components of arches rather than members in bending only. Another characteristic is that since some or all of the members must be considered in three dimensions, the connection problems are much greater and the efficiency of the structure may be lost through increased cost of fabrication.

There has been extensive development of plastic design of steel structures in the last few years, but comparatively little attention has been paid to development of space structures. The problem to be solved is the development of forms for which members and connections are relatively simple and inexpensive. The average engineer and architect does not have the time to sit down and study these types of structures and it would seem to be advantageous to the structural steel fabrication industry to assist him in this matter and to indoctrinate the architects in the acceptance of these new forms. The latter task is the least difficult of all the problems because the architects are eager to use new forms and may be even willing to pay a slight premium for unusual shapes.

The purpose of this article is to describe several types of space structures and compare them with more conventional solutions. It is characteristic of space structures that it is difficult to describe them with ordinary orthographic projection used for engineering drawings. The engineer and architect must think and draw in three dimensions either with isometric or perspective sketches or with models. The structures are described here by photographs of models made of balsa wood and glued together with Duco Cement. Many more models than are shown have been developed, and it is a very fascinating hobby to work on these models and to evolve new forms which satisfy all the criteria for economy, ease of design and beauty. We have not been quite as successful as we would like to be in getting our architect clients to use these structures and in this phase, we need full cooperation of the fabricators.

Square Domes (Figure 1)

The first model to be described is the series of square domes shown in Figure 1. The example to be studied has spans of 40 feet for the interior members. The diagonals form triangular cross arches which thrust against the horizontal tie beams forming the square. Note that these tie beams on interior spans are also supported at the center by the rafters acting as catenaries and have a span in bending of only 20 feet. In the model shown, the diagonals in one of the units have been omitted so that the effect of the diagonals can be demonstrated by the model. Note

Mr. Ketchum is a Partner in the firm of Ketchum, Konkel and Hastings, Consulting Engineers, Denver, Colo.

the columns have been used at 20 foot centers in the outside spans. This is normally no handicap for most structures.

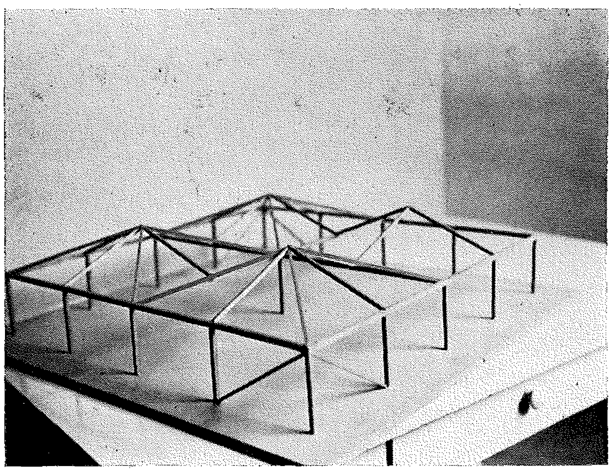

FIGURE 1

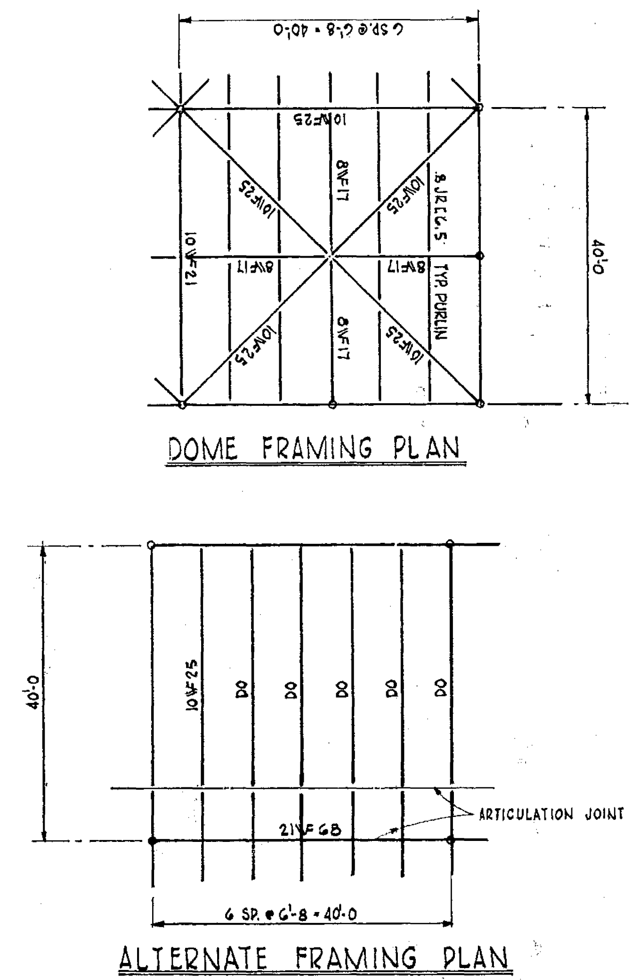

FIGURE 2

The analysis of this structure is perfectly straightforward and is statically determinate except for the horizontal members which act as two-span continuous beams. Some of the members have both bending and tension. A design was made for this structure, and the cost was compared with an orthodox articulated beam structure with the same spans. Framing plans for the alternates are shown in Figure 2 and typical connection details are sketched in Figure 3. The detail used at the crown of the dome is a separate crown block to which the eight members may be bolted. All of the blocks will be the same. The detail at the pipe column is a large cap plate to which the arches and ties are welded in the field.

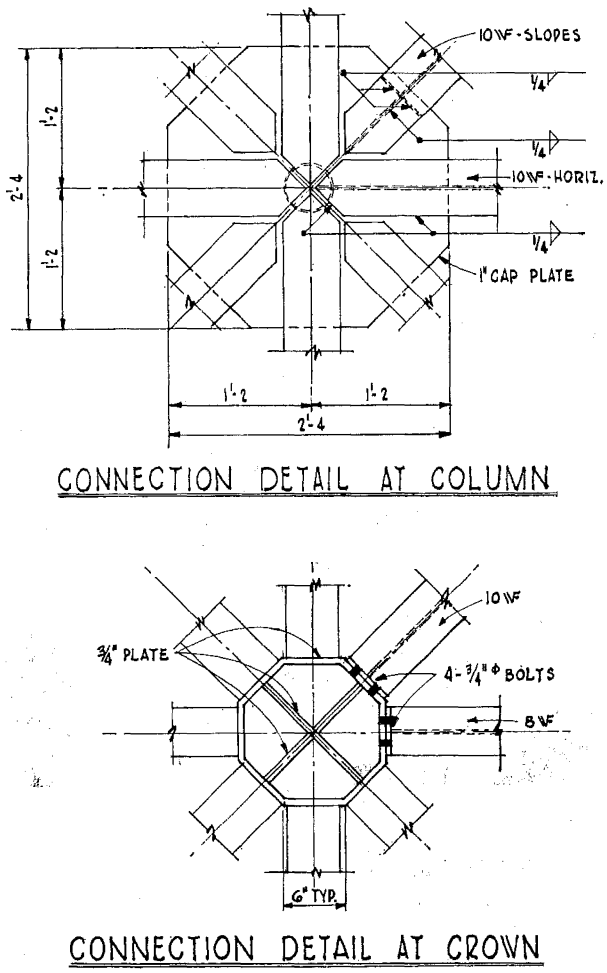

FIGURE 3

According to our calculations, the dome design has 8900 pounds of steel for a typical interior bay not including columns, and the beam and the articulated beam structure have a weight of 9150 pounds for an interior panel. The cost of fabrication of the articulated beam system may be lower so the dome design may cost a little more. Also, the cost of the wood deck might be greater because there is more area and more cutting. However, this example does demonstrate that this dome structure may be quite competitive with orthodox structures.

Arches With Cross Girders (Figure 4)

The next model of a space structure to be considered has a structural action which resembles that of a suspension bridge. A photograph is shown in Figure 4. There are two structural elements: (1) Thin circular arches, (2) Cross trusses or girders. The arches are hinged at the points where they are attached to the girders, act only as columns and do not take any bending forces.

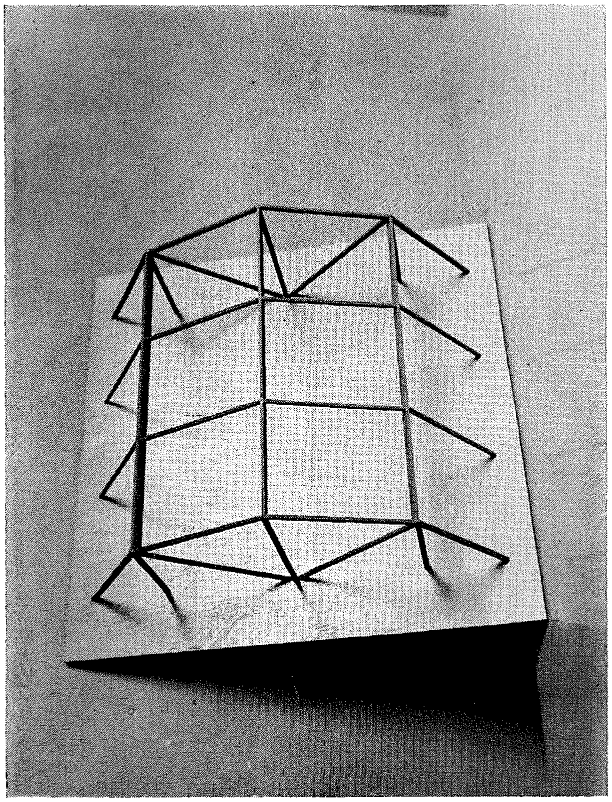

FIGURE 4

For full dead and live load, there is practically no stress in the cross girder. They come into use only when there is an unbalanced live load. Therefore, they are quite small in comparison to the size of truss required for the entire span. This design is most suitable for gymnasium structures where the side columns will not interfere with the performance of the structure as in the case of an aircraft hangar.

Arches With Trussed Diagonals (Figure 5)

The cross girders in some cases may be eliminated by placing diagonals in all the bays of the structure as demonstrated by the photograph of the model shown in

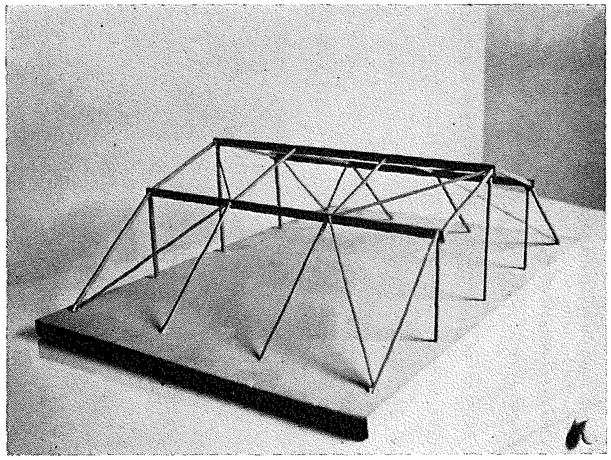

FIGURE 5

Figure 5. This structure may further reduce the material required for a gymnasium type structure. Double diagonals may be used instead of the single diagonals which would only be a little larger than the members required for wind bracing.

Pitched Trusses (Figures 6, 7 and 8)

A space structure has been designed and built with trusses tilted so that the vertical web members become purlins for the roof structure. A model of this type of structure is shown in Figure 6. It represents two bays of a typical truss building. The outside trusses are supported by columns along the lower edge. The web members of the trusses are arranged similar to members of a spandrel braced arch. The length of the diagonals is thereby reduced and a much more pleasing form is created. Economy studies were not made to compare this with ordinary construction, but it is believed that when all factors are considered this system will be competitive with the usual type of truss construction. It may appeal to architects or

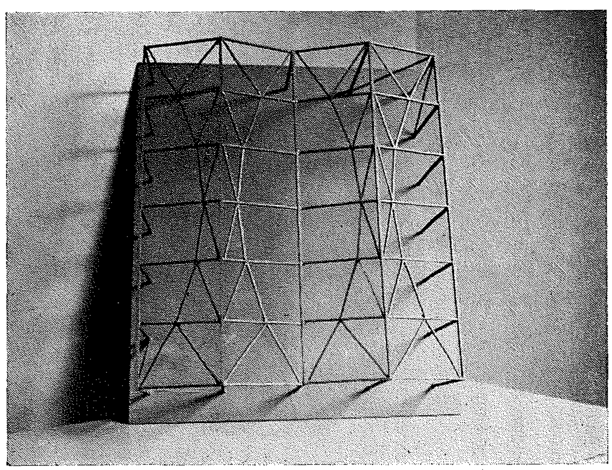

FIGURE 6

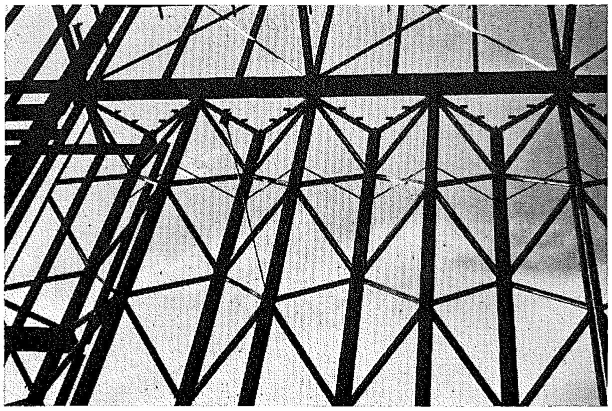

FIGURE 7

owners as being a more interesting and cleaner framing system.

Several enterprising structural engineers have built structures of this type in this country in recent years. In Figure 7 is a picture of trusses used in the Canon City, Colorado Court House. Nixon and Jones of Denver were the architects, and Johnson and Voiland of Denver were the structural engineers. The trusses are designed to take heavy mezzanine loads in several places.

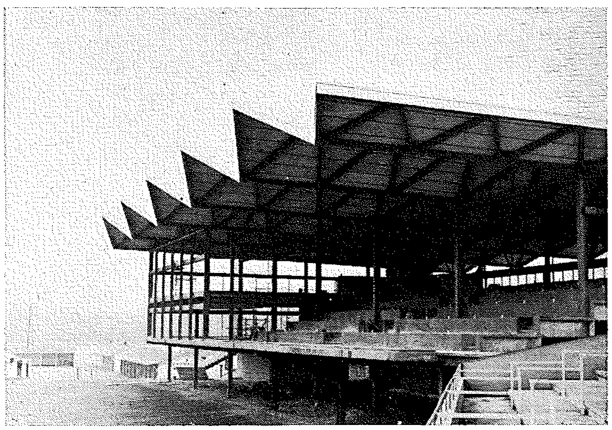

FIGURE 8

Another interesting example of this type of construction is the roof for the Foxboro Racetrack. The structure was designed by Abraham Woolf and Associates of Boston, Massachusetts and is shown in Figure 8.

Arches and Horizontal Trusses (Figure 9)

The last model of a space structure to be described is the series of arches shown in Figure 9. This structure would be most suitable for a church or other buildings requiring an unusual shape. The feature of this structure is that the arches, which in this case are of pyramidal shape, are set in the top of slender columns. The arch thrusts are carried by flat horizontal trusses to the ends of the structure where they are taken out by either diagonal members in the end walls or by horizontal ties between the two trusses. There is a lightness and elegance to the building that cannot be achieved by any other structure. The design of this structure is not difficult and is statically determinate. The inclined thrusts at first may be confusing to the uninitiated. Almost any form of arch or rigid frame may be used for this type of structure so the freedom to experiment is greatly enlarged.

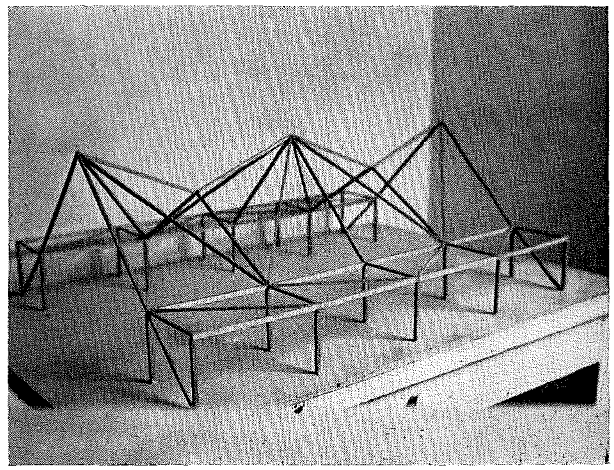

FIGURE 9

Conclusions

There are many other space structures that can be developed with a little ingenuity and perseverance. It is essential that they be studied by use of balsa wood models. It is especially important that the connection details be simplified so the saving in material is not eaten up by extra cost of fabrication. In many cases these interesting and beautiful structures will sell themselves regardless of cost and most architects would like to have these structures available as a solution of some of their problems.

Structural Steel After A Fire

F. H. DILL

A fire in a steel framed building or other steel structure almost inevitably leaves a nightmarish assortment of warped and crooked steel and a question of how much damage has been done to the strength of the steel.

The first thing necessary in developing an answer to the question is to set it apart from the subject of protecting steel from fire and from discussion of how to straighten warped and crooked steel. Means for protecting steel from fire are thoroughly discussed and detailed in the American Institute of Steel Construction booklet "Fire-Resistant Construction in Modern Steel-Framed Buildings." Methods for straightening steel by mechanical means, either in the shop or the field, are well known in the fabricating industry. Straightening with localized heating has been described in many articles in the technical press and a particularly good presentation of the subject was made by Mr. R. C. Stitt at the Institute's 1952 National Engineering Conference.

The question, "What does fire do to the strength of structural steel?" invites a brief review of how heating and cooling affect an as-rolled structural carbon steel such as ASTM-A7 steel. Shapes and plates in the as-rolled condition are the common material used in steel structures. Such steel usually has some residual stresses in it as a result of the rolling operation and final cooling. Subsequent heating and cooling may reduce these stresses or may simply redistribute them. In either case they are not generally given any special consideration in design calculations.

Heating structural carbon steel to stress relieving temperatures, 1100 to 1200°F, or to temperatures of 1500 to 1600°F, as in annealing or normalizing, and cooling it slowly may reduce its as-rolled tensile strength and yield point by as much as 5000 psi. These reductions of strength are well recognized, both technically and commercially, however, and usually do not affect design calculations.

Heating structural carbon steel to temperatures that are into or above its transformation temperature range, 1300 to 1550°F, and rapidly cooling it, as by quenching it in water, will of course harden it to some extent. There has not been much information published about this, largely because of lack of interest in the subject, but some tests have been made. Data from a few tests made with ⅝ and ¾ inch thick ASTM-A7 steel is provided in Figure 1. Other ASTM-A7 steel might not harden as much as these samples did. Very little of it would harden beyond the higher values listed.

Mr. Dill is Assistant to the President, Engineering Research, American Bridge Division, U. S. Steel Corp., Pittsburgh, Pa.

	As Rolled	*After Heating to 1475°F and Fully Quenching in Water*
Yield point	35,000 psi	53,000 to 77,000 psi
Tensile strength	63,000 psi	83,000 to 107,000 psi
Elongation in 8"	28%	15%
Brinell hardness	140	212 to 270

FIGURE 1

Heating the steel to very high temperatures, above 2000°F for example, and maintaining it at such temperature for half an hour or more may develop some grain coarsening and it will almost assuredly develop a very heavy scale on the steel unless the steel is well protected from the atmosphere. The grain coarsening by itself is not particularly detrimental to the steel. Some structural carbon steels have fairly coarse grain in their as-rolled condition without any impairment of their strength and ductility. It is only when the grain coarsening is accompanied by excessive scaling and oxidation that it becomes truly harmful to the steel. Fortunately, when steel is overheated to this extent it is usually recognizable as "burnt" steel.

A final characteristic of structural carbon steels that is as important to the question at hand as the others just discussed is the change of strength as temperature is increased. Yield point decreases with any increase of temperature. Tensile strength increases a little as temperature rises to about 400°F, then it decreases and falls below the atmospheric temperature tensile strength at all temperatures above 600 to 700°F. Modulus of elasticity is reduced as temperature increases. Approximate values for these properties of structural carbon steel at various temperatures, expressed as percentage of atmospheric temperature properties are noted in Figure 2.

Temperature	*Yield Strength*	*Tensile Strength*	*Modulus of Elasticity*
Atmospheric	100%	100%	100%
400°F	90	110	95
800	60	85	85
1050	50	50	80
1300	20	15	70
1600	10	10	50

FIGURE 2

To make these various characteristics of steel have any significance in judging the effects of a fire it is necessary to consider now what happens to the steel in a fire. Naturally the steel gets hot, but estimates of the temperature attained are often on the high side of actuality. Temperatures of 2000°F and sometimes even higher can be attained but temperatures of 500 to 1300°F are probably more common. Often the estimate of temperature attained is influenced by the amount of distortion in the steel without any realization that there is no correlation between the two factors. It is not recognized that stresses great enough to buckle the steel are developed at temperatures of only 250°F when the thermal expansion of the steel is completely restrained. At higher temperatures where expansion is greater and yield point is lower, even less restraint of the expansion will create sufficient stress to buckle the steel severely and permanently. The amount of expansion at fire temperatures is often surprising, too. It is an inch in each ten feet of length at 1100°F and it is one percent of the length at a temperature of 1300°F.

The effects of loads that are on the steel while it is hot are usually more easily visualized than the effects of stresses that are created by the expansion of the steel. It is quite apparent that a beam designed to support its loads at a stress of 20,000 psi will begin to sag when the temperature reaches 1000°F and the yield strength of the steel drops to perhaps 18,000 psi. In contrast to this, it is only sections of low bending strength, like angles or bars, or pieces that are heated to very high temperature that truly sag by their own weight. Restraint of expansion and interaction from connected members are more predominant causes of deformation of unloaded members.

Another cause of much of the distortion that is found in steel that has been through a fire is the unequal and unsymmetrical heating that takes place. Mr. Stitt's presentation on straightening and forming by the use of locally applied heat that was previously mentioned explains the mechanisms by which the deformations are developed. It is necessary here to add only the comment that these deformations, like the development of buckling, can occur at surprisingly low temperatures. Permanent deformations can be developed by temperature differences of as little as 450°F.

The cooling of steel from its fire temperatures is important too. Much of it cools in air and some may be cooled very slowly, as under a covering of hot debris. Some of the steel, of course, may be cooled by the water from fire hoses. This is spectacular cooling, but from a metallurgical standpoint it is often much less drastic than the water quenching that is used in heat treating and, of course, it has no metallurgical effect if the steel is below its transformation temperature when the water strikes it. From a practical viewpoint, cooling by the water from fire hoses usually leaves its mark by making the water cooled pieces some of the most distorted pieces in the structure. This becomes understandable when it is recalled that men who do heat straightening like to use water spray or water poured onto the steel to get sharp bends that are difficult to obtain without such treatment.

The point of these several observations of what happens during a fire is to emphasize that very gross deformations of the steel take place under the influence of temperatures that are below the transformation temperatures of the steel and in the range where heating and cooling rates have no appreciable metallurgical effect on the steel. Large deformations are not in themselves evidence that the steel has been heated to high temperature. They result from stresses imposed on the steel while it is at reduced strength. The same kind of deformations would result if comparable stresses were applied to the steel while it was cooling from its final hot rolling.

Tests of structural carbon steel that has been severely distorted in a fire offer some corroboration of this. There have been many tests of steel that has been exposed to a fire and the results of various tests do not differ greatly. The data in Figure 3 is taken from samples representing the most distorted beams and channels from a building which suffered a fire during construction. The specimens were from the webs of the sections; the usual test location.

Section	*Yield Point*	*Tensile Strength*	*Elongation in 8″*
6″ channel	33,250	57,700	23.6
6WF15.5	36,100	57,600	28.7
8BL13	33,450	53,250	25.4
10BL15	37,850	59,880	27.4
12I20.7	36,200	62,600	27.0
12WF16.5	38,980	59,050	31.5
12WF31	40,450	62,900	28.8
16CB40	35,580	62,150	29.2
Average	36,480	59,390	27.7

FIGURE 3

None of the yield points in these tests was below the specification minimum of 33,000 psi. Only one of the tensile strengths was as much as 10% below specification requirements (60,000 psi). Even this probably would not cause any concern to a designer who is basing the factor of safety on yield point conditions. It may be noted also that these were light sections which, because of normally low finishing temperature in rolling, are generally more vulnerable than heavier sections to loss of strength by re-heating after rolling.

The conditions that result from very high temperatures during a fire need some attention too. Unless there is some protection from the atmosphere, any steel that has been heated to a very high temperature will be very heavily scaled and possibly eroded. Both of these actions will reduce the cross-section of the material and their damage is easily evaluated by measurement. Grain coarsening and severe oxidation that can result from high temperatures may be damaging to the steel also but these are things that take time to develop. If they occur it is almost certain that other conditions, such as severe distortion or the loss of section from scaling or erosion, will already have caused rejection of the steel as unfit for further use.

The question of whether steel has been hardened by drastic cooling during a fire has been partly answered by prior comments on temperatures attained, usual rates of cooling and information about the response of structural carbon steel to drastic cooling. One further comment can be offered. That is that hardness testing with a portable Brinell hardness tester can quickly resolve any doubts that may exist after the various other means of judging the steel have been applied. Hardness tests can, of course, pick out steel that may have been unduly softened by the fire as well as that which may have been hardened. ASTM-A7 steel is expected to have Brinell hardness in the range of 120 to 180. Hardness measured after the steel has been in a fire that are below 100 or above 200 would be warnings that the material under examination needs more scrutiny before it may be continued in service.

Connections have not been given any separate consideration in this discussion, because the material in connections is subject to all the phenomena that have been reviewed. The greatest effect of a fire as far as connections are concerned is the overstress that it imposes on them. This often loosens rivets or bolts and makes their replacement necessary. In welded connections it makes careful inspection necessary to make sure that no welds have fractured as a result of excessive stresses.

Conclusions

The conclusions from these various considerations of the effects of fire on structural carbon steel are that even though the fire will almost certainly warp and twist the steel, it does not inevitably follow that the strength of the steel is reduced. It is almost certain that any steel which has been heated hot enough to undergo damaging grain coarsening or which has been cooled rapidly enough to harden it will be so badly distorted that it would have no consideration for re-use anyway. This leads to a general statement that:

> Steel which has been through a fire but which can be made dimensionally re-usable by straightening with the methods that are available may be continued in use with full expectance of performance in accordance with its specified mechanical properties.

If this rule needs further support it may be found in the observation that straightening operations themselves are tests of the strength and ductility of the steel.

Discussion

(*Q*) S. Clark, AISC: Does the heavy scale (which you described as possibly being accompanied by grain growth) have a distinctive color? If not, how can it be identified?

(*A*) Dill: The scale would normally be dark grey to black, but it often picks up coloring from surrounding materials. Its color is not particularly significant. The thickness of the scale and the pitting that would be with it, indicating what the blacksmith would identify as "burnt" steel, are the warning signs.

Steel Framed Houses in the West

SAMUEL H. CLARK

It is a pretty safe bet that nearly everyone is interested in the design of houses. Unless you happen to live in an apartment or in a tent, this must apply to everyone who needs shelter. If you're not interested in the design of houses, most likely your wife is. Houses with steel frames are becoming increasingly important in these United States. When I chose the title, "Steel Framed Houses in the West," I had not anticipated the number of examples I could find East of the Mississippi. It looks as if everything is included in the term "West" except the State of Maine, and that would be Down East to a Bostonian. I hope to give you some good reasons why steel framed houses are important and then I would like to show some examples of how they are being built.

The importance of steel framed design for houses certainly is something of interest to owners or prospective owners. Architects are also interested—so are general contractors or builders. So are steel fabricators and so, last of all, are lending institutions.

Excluding, perhaps, people who are not interested in contemporary houses or contemporary living, or who are not interested in quality construction in a residence, I feel sure that almost everyone is interested in steel framed houses.

Owners Like Steel Framed Houses

Whether it's an impossible site on a steep hillside that makes it hard to develop the lot, or whether it's a desire for a long overhang on the roof, a steel framed house offers a solution which many owners have accepted with enthusiasm. The use of large amounts of glass for modern "indoor-outdoor" living leads to more and more requirements for high strength and light crisp appearance. Here is where steel framing has it all over the competition.

One big consideration that impresses home owners is the cost of maintaining their new house. In this question steel framed houses pay dividends over the years. Problems of patching of plaster where warped timber beams have opened up cracks and the costs of termite protection all are eliminated when steel framing is used. Steel framing will not burn. This is a factor to be considered when thinking of fire insurance rates and the loan value of a home. Having a house with incombustible framing provides a measure of safety and security worth a great deal to a home owner.

One of the active companies in promoting the use of steel framing in residential construction has been Bethlehem Steel Company. They recently received a deluge of letters about a series of advertisements in West Coast newspapers. Among the comments in some of the letters are the following quotations: "We are planning on doing some remodeling on our home. Headers are needed for the sliding glass doors so will appreciate your free booklet." "I am planning to build a home and your ad in the Tribune interests me." "If it is possible I would like you to send me a list of builders who use steel as their chief construction material for homes." "I have been interested in building a steel house and saw one in the San Diego Union Daily." "I am planning to build a home on a steep hillside lot. Saw your advertising on steel framing and its advantages." "A list of architects designing for steel framing would also be greatly appreciated." "Will you please send us some interesting examples of how and why architects and builders are at long last awakening to the fact that more steel should be used in the construction of a good house. They are undoubtedly learning this from home owners who, like ourselves, have found that wood joists, even at 12″ on centers, cannot carry the load of a second story without additional cross support of the steel I beam." "After a recent experience with pests and water damage to the framing of some real estate of mine, I find myself quite receptive to suggestions for a more permanent type of construction."

Architects Like to Design in Steel

Many architects have found that the use of steel framing frees the design of a house. The possibilities are widened for use of long interior spans, curtain walls, such things as hillside platforms, or even anchored cantilevers. The advantages of wide open feeling toward the designer can be achieved with steel framing to an extent not possible with other material. The "light look" is desirable in many cases and can be attained most easily with steel. The use of modular design lends itself to the prefabrication techniques of steel framing also. Possibilities of welded frames and more clean-cut details and connections open up the use of exposed framing and give the architect a new medium of expressing the most graceful and suitable finish detail he might conceive. A great amount of flexibility is possible in enclosing space with different shapes of buildings and a precise appearance of framing is also desirable in many cases.

Engineers Are Enthusiastic About Steel Framing

Foundation problems on steep sloping sites can be solved more easily with steel framing, whether it is a house or an apartment, than with other types of design. With the "wide open spaces" rapidly disappearing in many cities, the most abundant remaining property seems to all be on marginal steep hillsides. Access to these sites

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is difficult and conventional continuous wall footings with stud wall underpinnings become extremely expensive when added to the large amounts of earth work. For these reasons, and also because of dangerous possible slides in many locations, the use of steel frame underpinnings on a steep hillside lot has been on the increase. One engineer tells me that this design has made the difference between being able to build or not being able to use a given steep lot when the lighter weight and more economical steel framing can be used. A builder in Alameda County, California has laid out many lots for steel framing based on a design which involves drilled in place concrete piling and a minimum of earth work. On top of this is erected the steel frame.

Steel frame underpinnings do not necessarily have to go on steep lots either. The Watson Foundation Company in Fort Worth, Texas has conceived an innovation which uses steel framing on concrete caissons on level ground. In this case, the elimination of wall cracks due to settlement and problems of movement of the subsoil due to freezing, thawing and changing of moisture content were all solved by the use of steel framing and these concrete caissons which extend down to bedrock. Actually, there is nothing new about these principles or ideas for larger buildings. They are only new in their application to residential construction.

The ability to take care of lateral loads by means of rigid frame design and not by shear walls is another advantage of steel framing in houses. This idea, of course, is used in many commercial and industrial buildings also. It is interesting to note that one of the foremost research experts on earthquake vibrations, Professor Lydak Jacobsen at Stanford University, is just now having a new steel framed house designed for his own.

There is no need to dwell on the possibilities of longer spans with steel, on the shallower depths of structural framing in a roof or on the efficient use of material and strong connections with a more resilient integrated framework. These advantages of steel over other materials are well known to all of you here, I'm sure.

General Contractors and Builders Like to Work With Steel

The fact that the basic material is a manufactured item holds many advantages for a general contractor or a builder when steel framing is used. This basic stock material is fabricated in a shop away from the building site so that the builder has only to schedule the relatively short erection period to coordinate with the work of other trades. This reduces on-site labor and provides a basic framework from which to work. In addition to this the builder immediately has a platform to work on when the structural frame is erected on a steep hillside.

Fabricators Are Interested in Steel Framed Houses

Many smaller fabricators are recognizing that this is a possibility of a new market for their steel framing. Certainly there are many shops who would not be interested, but the fact that I am speaking here today is evidence that some shops are impressed with the importance of the steel framed house. The development of such a market will admittedly take some careful planning and a great deal more time and effort and expense than has already been put forth.

Lending Agencies Are Interested in Steel Framed Houses

The banks or savings and loan companies or insurance companies who are underwriting new custom built houses cannot help but be impressed by the facts. Here is a material which will not be subject to termite or rodent attacks, that will not support combustion, that has stability and does not change properties with time, will not warp, check, shrink, rot or tire. As a matter of fact, inquiries have come from several banks and savings and loan companies to Bethlehem Steel Company in San Francisco about their advertisements for steel framed houses.

Even though it means "breaking with tradition" in house design, many of the more forward lending agencies have recognized the basic advantages of steel framed houses. One of the prime considerations in making loans is to make sure the house does not slide away in the event of a bad rainstorm. Steel framing with a minimum of earth work on a steep hillside offers an answer to this. A photograph I have seen shows a mud slide extending through the steel framing and the underpinning of a house in the Los Angeles area. The house remained firm but the slide went through.

Examples

Now I would like to show some of the recent examples which illustrate some of the advantages of steel framing in houses.

Figures 1 and 2 show a steel framed family retreat,

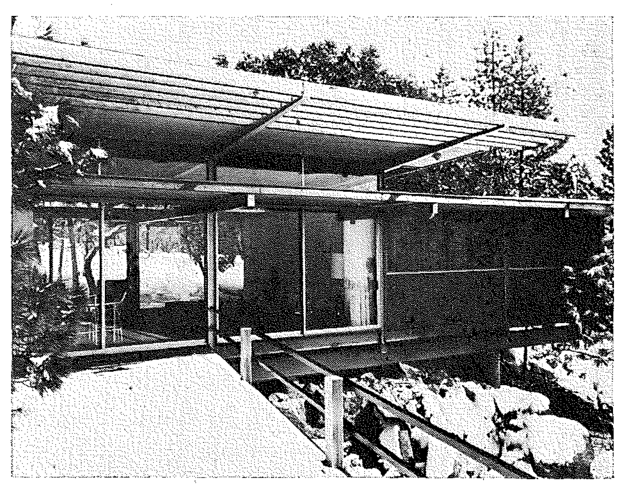

FIGURE 1

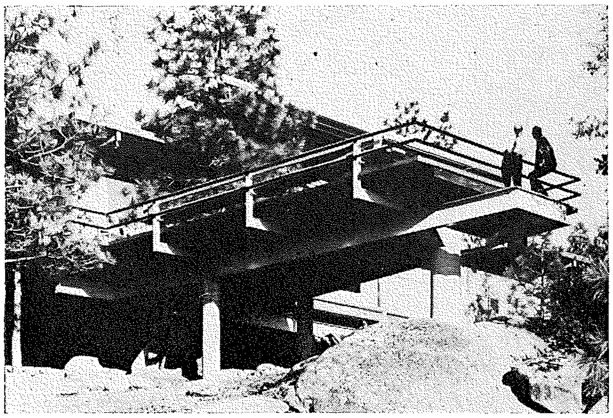

FIGURE 2

designed for H. D. Bartlett and his son, Paul, of Fresno, California by Architect David Thorne of Berkeley. This two bedroom, fully equipped home, is located an hour and a half's drive from Fresno in the Huntington Lake area and provides a striking contrast to its natural surroundings. To set the house securely on this rocky point where a view of the entire valley is visible over the treetops, the architect used a steel frame to carry all structural loads of the house and open up the interior to take full advantage of the view. Only one small cedar tree was removed to set the foundation. The entire 1200 ft. floor area is carried on two steel beams, 60 ft. long, supported at two points over concrete pylons keyed into rocks. These 16 WF 36 beams cantilever the rectangular shell of the house 12 ft. in each direction. Floor and roof areas of the house are framed with six steel bents or frames composed of 5 WF 16 columns and 8B15 beams spanning 17 ft. with a $5\frac{1}{2}$ ft. cantilever over each column. Interior paneling in the living room is book matched panels of teak. The floor is white vinyl tile. The deck was cradled between inverted frames composed of 8B15 sections and 4 WF 13 columns. These short columns serve as railing posts for 2 x 2 square steel tubing. On one side of the house a circular steel stairway serves as access for the flat roof portion of the house, which is used as a sun deck in summer. Instead of pulling out or destroying the natural growth, Thorne built deck areas and overhangs around existing trees. An opening in the roof overhang permits a small cedar to grow up through the roof line.

Architect Raphael Soriano solved a hillside site problem with an award winning steel framed house (Fig. 3) for the McCauley family in Mill Valley, California. Natural foliage and magnificent trees were preserved, and glass walls and doors dramatize the hillside view. The home, built over a steep site, utilizes a steel frame every 10 ft. The clean, crisp lines of the steel are painted a warm color giving emphasis to the modular design and a carefree elegance to the structure. Folding doors supported by steel frames can be opened to give additional floor area off the living room.

Architect David Thorne added a splash of the spectacular to this home on a bluff in El Cerrito, California (Fig. 4). Taking full advantage of the lot's view possibilities he designed the living room kitchen wing of the house with an 8 ft. cantilever over the steepest portion of the lot. Main support of the roof is rendered by four steel carrier beams spanning 72 ft. between concrete block shear columns. Glass non-bearing curtains walls on three

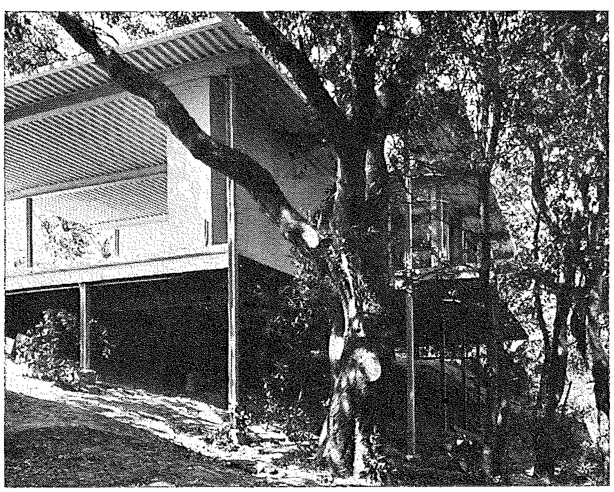

FIGURE 3

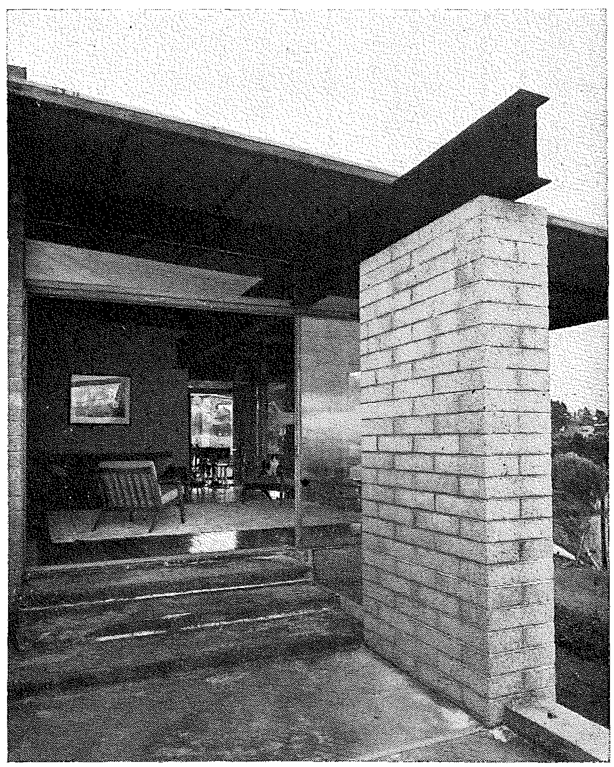

FIGURE 4

sides enhance the view by offering a 180 degree panorama. This is without any pillar or wall obstructions. The overhang achieves an 11 ft. cantilever to screen the area from daytime sun.

Rigid connections as strong as the steel members themselves provide thinness of line and a crisp elegance to this design for the Fields family by Craig Ellwood of Los Angeles (Fig. 5). The steel framework is composed of tubing in two sizes. It is all welded. Connections are precise and clean. Modular arrangement was worked out so that prefabricated panel units could be installed as well as door units, windows and jalousies.

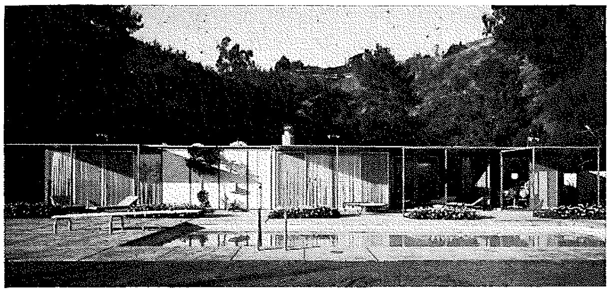

FIGURE 5

The Whelan house (Fig. 6) in Menlo Park, California, shows how adobe bricks can be used with steel framing to achieve both an interior and exterior finish surface for the walls at the same time. The design was by Donald Knorr.

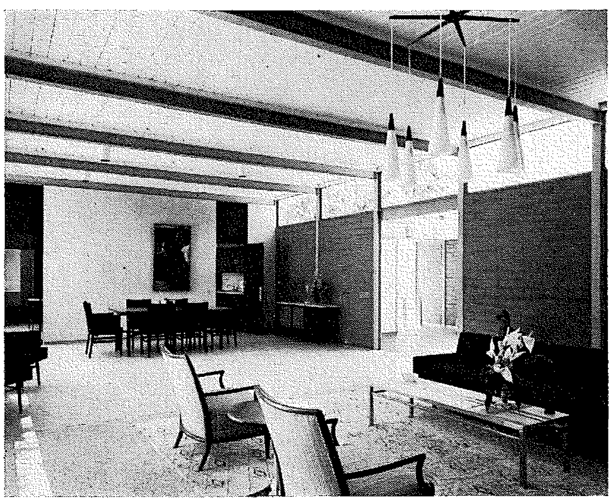

FIGURE 6

Figure 7 illustrates a home on one of those "impossible" lots in Sausalito, California, just north of San Francisco. Engineer John Brown says, "Using the steel frame solved the costly problem of underpinning the house to elevate it to street level and the shorter time required for the steel work made the overall cost of the

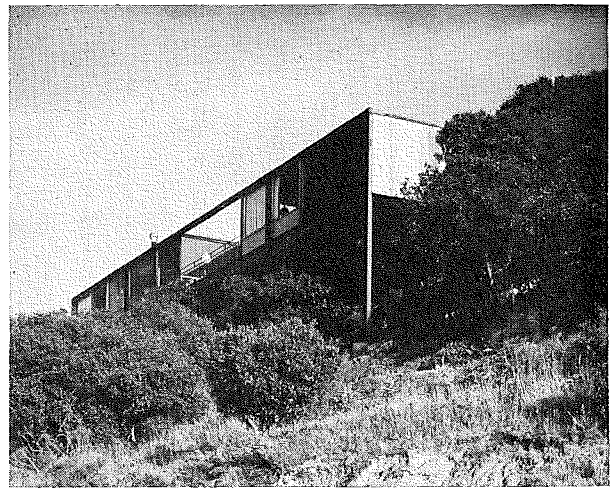

FIGURE 7

home lower." This 2200 sq. ft. house was built for architect Hoop's own family for $24,000.

A good two story house is hard to find these days, according to "Better Homes and Gardens." This home for the Leon Grossier family (Fig. 8) won the magazine's 5 star award in 1958. Steel framing was used. Location, Lexington, Massachusetts.

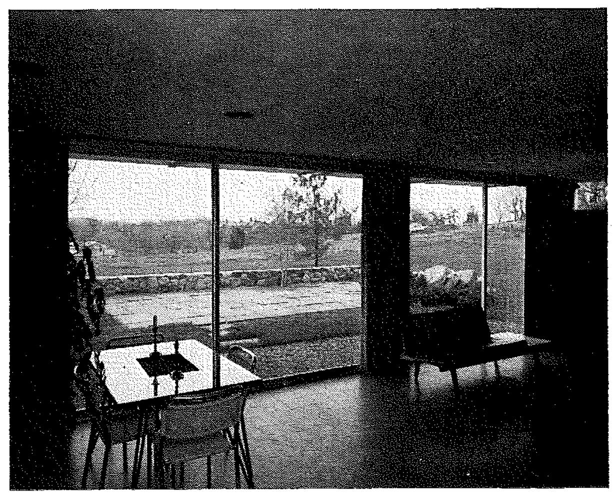

FIGURE 8

The completely modern house shown in Figure 9 was described in Look Magazine, March 15, 1960. It is located in Rye, New York, and was designed by Ulrich Franzen for the George Weissman family. To quote the owner, "It turned out so great, I nearly forgot the mortgage." This home has a steel frame.

Steelwork was left exposed throughout the award winning W. C. Bailey home in Los Angeles (Fig. 10). With a steel frame, Architect Pierre Koenig kept his interior plan completely open. This divider wall separates built-in kitchen utilities from the living room. Floor is white

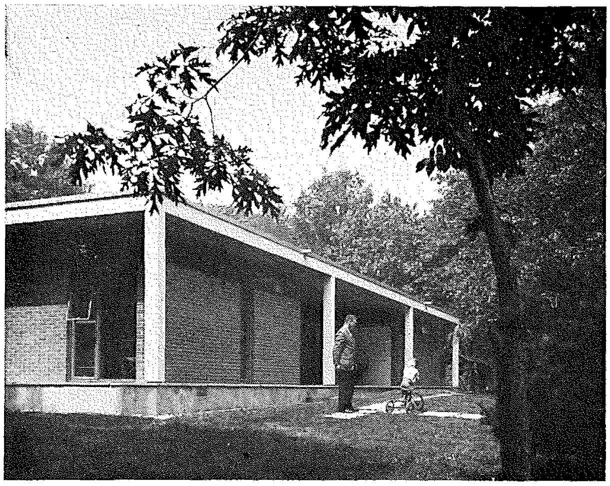

FIGURE 9

vinyl tile and walls have vinyl paint. Structural steel is painted charcoal black. Furnishings repeat this theme in stainless steel, black leather and soft-toned eggshell carpet.

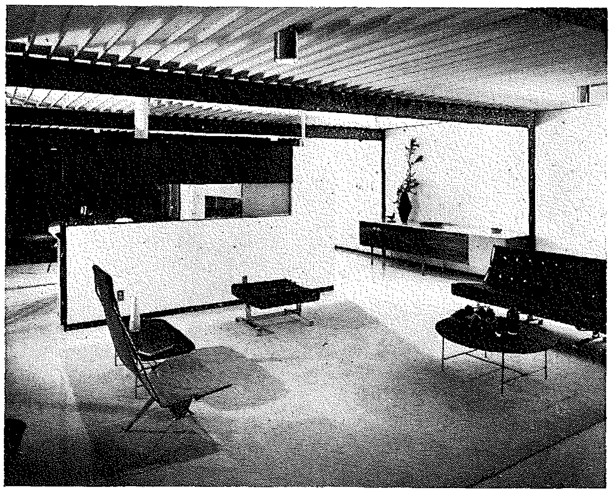

FIGURE 10

Architects Johnson & Hawley, A.I.A., of Palo Alto left the steelwork exposed in the living room of this home for partner Milton Johnson (Fig. 11). Glass panels close space between partition wall separating kitchen and give visual depth to the span of steel.

Architect David Thorne lifted 3,000 sq. ft. of house on five fingers of steel to transform a huge rock into a view lot in the Oakland, California hills (Fig. 12). Jazz pianist Dave Brubeck, his wife and four children, have privacy, isolation both for jam sessions and the needs of a nursery, plus plenty of safe deck space as play area for the children. The structure has the appearance of growing from solid rock, yet its spectacular cantilever gives an impression of soaring eloquence.

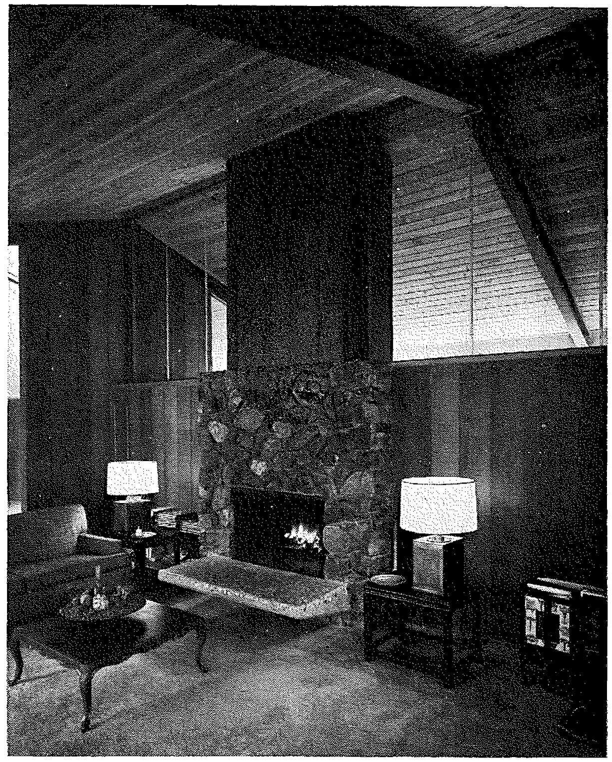

FIGURE 11

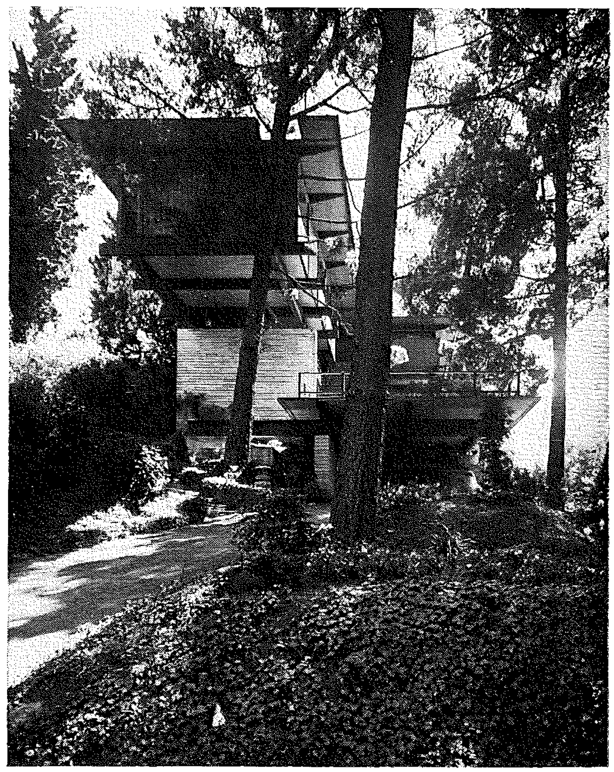

FIGURE 12

Registration List

COMPANY	CITY	INDIVIDUAL
ACORN IRON WORKS, INC.	Detroit, Mich.	ROBERT D. SALLEN
ALLEN STEEL COMPANY	Salt Lake City, Utah	ROBERT B. ALLEN
	Salt Lake City, Utah	VERNON BRADY
	Salt Lake City, Utah	W. C. HOWE, JR.
	Salt Lake City, Utah	ROBERT W. TRAXLER
ALLIED STRUCTURAL STEEL COMPANIES		
MIDLAND STRUCTURAL STEEL CORP.	Hammond, Ind.	E. C. BROCKE
	Hammond, Ind.	R. S. CLARK
ALLISON STEEL MANUFACTURING CO.	Phoenix, Ariz.	J. AUGUST RAU
AMERICAN BRIDGE DIVISION—U. S. STEEL CORP.	Birmingham, Ala.	G. P. WILLOUGHBY
	Chicago, Ill.	W. K. MCGRATH
	Denver, Colo.	R. A. ELK
	Denver, Colo.	W. R. HOLMSTROM
	Los Angeles, Calif.	G. P. WILLARD
	New York, N. Y.	C. I. ORR
	Orange, Texas	O. H. LEBLANC
	Pittsburgh, Pa.	K. D. CUNNINGHAM
	Pittsburgh, Pa.	O. C. DIEHL
	Pittsburgh, Pa.	F. H. DILL
	Pittsburgh, Pa.	F. K. GOODELL
	Pittsburgh, Pa.	A. S. MARVIN
	Pittsburgh, Pa.	R. A. SHAW
	Pittsburgh, Pa.	R. G. WALLIN
AMERICAN SMELTING & REFINING CO.	Denver, Colo.	R. D. THOMAS
ANDERSEN & KOERWITZ	Denver, Colo.	R. A. ANDERSEN
	Denver, Colo.	L. C. KOERWITZ
ARKANSAS FOUNDRY COMPANY	Little Rock, Ark.	HAROLD ENGSTROM
	Little Rock, Ark.	ROBERT F. OATES
ARKANSAS STATE HIGHWAY DEPARTMENT	Little Rock, Ark.	W. E. ROWND
ARKANSAS, UNIVERSITY OF	Fayetteville, Ark	PROF. C. W. YANTES
ATCHISON KLOUERSTROM	Denver, Colo.	D. A. KLOUERSTROM
AUSTIN BROTHERS STEEL CO.	Dallas, Texas	ARMO BASSE
BAKER & COMPANY, HUGH J.	Indianapolis, Ind.	L. G. BROCK
	Indianapolis, Ind.	WM. KOSSEBAUM
BELMONT IRON WORKS, THE	Philadelphia, Pa.	WM. E. HENDRICKS
BETHLEHEM FABRICATORS, INC.	Bethlehem, Pa.	ERNEST O. GERLACH
BETHLEHEM STEEL COMPANY	Bethlehem, Pa.	KARL DEVRIES
	Bethlehem, Pa.	W. H. JAMESON
	Bethlehem, Pa.	JOHN R. LECRON
	Bethlehem, Pa.	R. E. WILMOT
	Chicago, Ill.	L. D. HERB
	Chicago, Ill.	T. W. SPILMAN
BETHLEHEM STEEL COMPANY		
PACIFIC COAST DIVISION	Alameda, Calif.	W. E. EDWARDS
	Torrance, Calif.	R. W. BINDER
	Torrance, Calif.	L. A. NAPPER
BINGHAMTON STEEL & FABRICATING CO., INC.	Binghamton, N. Y.	JEROME A. PATTERSON
BOLAND, HENRY J.	Denver, Colo.	
BUELL, & CO., T. H.	Denver, Colo.	W. WEAVER

REGISTRATION LIST

COMPANY	CITY	INDIVIDUAL
Builders Steel Company	No. Kansas City, Mo.	R. T. Bruce
	No. Kansas City, Mo.	Gordon Finch
	No. Kansas City, Mo.	A. W. Templin
Burkhardt Steel Company	Denver, Colo.	Alvin Baumgartel
	Denver, Colo.	Ed D'Amico
	Denver, Colo.	C. W. Fischer, Jr.
	Denver, Colo.	J. D. Horgan
	Denver, Colo.	Walter E. Houghton
	Denver, Colo.	H. P. Krapp
	Denver, Colo.	Duane T. Molthop
	Denver, Colo.	Peter Molthop
	Denver, Colo.	Wm. D. Potter
	Denver, Colo.	J. E. Rowland
	Denver, Colo.	J. C. Schreiner
	Denver, Colo.	H. L. Welsh
	Denver, Colo.	Ed Wolfe
Butler Manufacturing Company	Kansas City, Mo.	Harold G. Simpson
	Kansas City, Mo.	Bryant W. Sparks
California Department of Public Works	Sacramento, Calif.	Arthur L. Elliott
California, University of	Berkeley, Calif.	Prof. Boris Bresle
Campbell Steel Company	San Antonio, Texas	R. Trent Campbell
	San Antonio, Texas	Roy D. Sprague
	San Antonio, Texas	Eugene E. Stocking
Canadian Institute of Steel Construction, Inc.	Montreal, Canada	R. E. David
	Toronto, Canada	D. L. Tarlton
	Toronto, Canada	D. K. Turner
	Vancouver, B. C., Canada	J. Wheeler
Capitol Steel & Iron Company	Oklahoma City, Okla.	A. F. Hansman
	Oklahoma City, Okla.	J. A. Heagy, Jr.
Carolina Steel Corporation	Greensboro, N. C.	Maxon H. King
Churchman Co., Inc., M. S.	Indianapolis, Ind.	Marvin Ferguson
Colorado Builders Supply Co., The	Denver Colo.	M. W. Jackson
Colorado Department of Highways	Denver Colo.	A. D. Newbold
	Denver Colo.	A. Zulian
Colorado Fuel & Iron Corp.	Denver Colo.	J. R. Caten
	Pueblo, Colo.	J. Shank
	Pueblo, Colo.	H. H. Stracy
Colorado School of Mines	Golden, Colo.	J. S. Johnstone
Colorado, University of	Boulder, Colo.	C. H. Bowes
	Boulder, Colo.	J. Chinn
Colorado State, University of	Fort Collins, Colo.	J. W. N. Fead
	Fort Collins, Colo.	J. R. Goodman
	Fort Collins, Colo.	R. W. Hayman
Columbia Geneva Steel Division, U. S. Steel Corp.	Denver, Colo.	H. A. Archer
Concrete Steel Corporation	Detroit, Mich.	Wm. Rose
	Detroit, Mich.	Fred Hirtzel
Denver & Rio Grand Western R.R. Co.	Denver, Colo.	A. G. Cudworth
Dave Steel Corp.	Lebanon, Ohio	Charles M. Hacker
Delaney, C. S.	Denver, Colo.	J. Linger
Denver Board of Education	Denver, Colo.	G. R. Miller
Denver Steel & Iron Works Co.	Denver, Colo.	M. L. Dye
	Denver, Colo.	D. F. Guell
	Denver, Colo.	S. Skalicky
	Denver, Colo.	J. G. Wilson
	Denver, Colo.	H. H. Wolleson

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DENVER, UNIVERSITY OF	Denver, Colo.	D. A. DAY
	Denver, Colo.	J. R. ORRIS
	Denver, Colo.	W. H. PARKS
	Denver, Colo.	B. WYLIE
DIETRICH BROTHERS, INC.	Baltimore, Md.	H. BUCKLEY DIETRICH
	Baltimore, Md.	H. W. DIETRICH, JR.
	Baltimore, Md.	EDWARD J. KLAUENBERG
	Baltimore, Md.	LLOYD E. LECOMPTE
DOLLINGER, JR., INC., JOHN	Beaumont, Texas	C. R. DOLLINGER
	Beaumont, Texas	H. SILVERMAN
ELKHART BRIDGE & IRON CO., THE	Elkhart, Ind.	F. E. MILLER
FLINT STEEL CORPORATION	Tulsa, Okla.	C. B. GANNOWAY, JR.
	Tulsa, Okla.	O. B. PEDERSON
FLORIDA STEEL CORPORATION	Tampa, Fla.	GEORGE KARRAN
FORBATH, E. F.	Rifle, Colo.	
FORT PITT BRIDGE WORKS	Pittsburgh, Pa.	C. K. BUELL, JR.
	Pittsburgh, Pa.	J. A. DONNELLY
	Pittsburgh, Pa.	F. E. EBERLE
GARVER & GARVER, INC.	Little Rock, Ark.	MARK GARVER
GATE CITY STEEL, INC.	Boise, Idaho	J. D. GRIFFITHS
	Omaha, Neb.	H. G. VOLLMER
	Wheatridge, Colo.	D. E. GELL
GATES RUBBER COMPANY	Denver Colo.	C. A. PAYNE
GENERAL STEEL COMPANY	Fort Worth, Texas	DOYLE E. MILLER
	Fort Worth, Texas	HERBERT G. WALKER, JR.
GOLDEN GATE IRON WORKS, INC.	San Francisco, Calif.	ARTHUR HOFFMAN
GRAND IRON WORKS, INC.	New York, N. Y.	S. D. KAPELSOHN
GREGORY INDUSTRIES, INC.	Lorain, Ohio	ROBERT C. FRIEDLY
	Lorain, Ohio	ROBERT C. SINGLETON
GROISSER & SHLAGER IRON WORKS	Somerville, Mass.	A. E. SHLAGER
GUIBERT STEEL COMPANY	Pittsburgh, Pa.	O. E. GUIBERT, JR.
HARTLEY BOILER WORKS, INC.	Montgomery, Ala.	EARL C. GREEN, JR.
HASSENSTEIN STEEL COMPANY	Sioux Falls, S. D.	HOWARD T. CASHMAN
HAVEN-BUSCH COMPANY	Grandville, Mich.	JOHN H. BUSCH
	Grandville, Mich.	KENNETH M. SWEERS
ILLINOIS, UNIVERSITY OF	Urbana, Ill.	PROF. E. H. GAYLORD
INDIANA BRIDGE COMPANY, INC.	Muncie, Ind.	H. E. WRAY
	Muncie, Ind.	G. C. BIRT
INDUSTRIAL STEEL PRODUCTS CO., INC.	Shreveport, La.	JOE L. DAVIS, III
INTERNATIONAL STEEL COMPANY	Evansville, Ind.	LINTON B. BURR
	Evansville, Ind.	FRED DULL
	Evansville, Ind.	JAMES B. IGLEHEART
	Evansville, Ind.	JOHN KOCH
IOWA STATE UNIVERSITY	Ames, Iowa	DR. CARL E. ECKBERG
	Ames, Iowa	PROF. DAVID A. VANHORN
JOHNSON & VOILAND	Denver, Colo.	R. H. VOILAND
JOHNSON, CLIFFORD & ASSOC.	Denver, Colo.	C. JOHNSON
	Denver, Colo.	P. A. UPP
JOHNSON, J. H. ASSOCIATES ARCH.	Denver, Colo.	J. H. JOHNSON
JONES & LAUGHLIN STEEL CORPORATION	Pittsburgh, Pa.	A. F. WEISE
JUDSON PACIFIC-MURPHY CORPORATION		
DIVISION YUBA CONSOLIDATED INDUSTRIES, INC.	Emeryville, Calif.	J. PHILIP MURPHY
KAISER STEEL CORPORATION	Los Angeles, Calif.	ALVARO L. COLLIN
	Oakland, Calif.	E. R. BABYLON

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Kansas City Structural Steel Company	Denver, Colo.	N. W. Funk
	Kansas City, Kansas	J. A. Hall
	Kansas City, Kansas	G. F. Tilden
Kansas, University of	Lawrence, Kansas	Prof. G. W. Bradshaw
	Lawrence, Kansas	Prof. D. D. Haines
	Manhattan, Kansas	R. F. Morse
Ketchum, Konkel & Hastings	Denver, Colo.	Milo S. Ketchum
	Denver, Colo.	R. F. Besier
Kline Iron & Steel Co., The	Columbia, S. C.	Furman L. Anderson, Jr.
	Columbia, S. C.	Wesley F. Thompson, Jr.
Lakeside Bridge & Steel Co.	Milwaukee, Wisc.	Alfred V. Jurevics
Lamson & Sessions Co., The	Cleveland, Ohio	A. E. R. Peterka
Leckenby Structural Steel Co.	Seattle, Wash.	B. E. Weeks
Lehigh Construction Co.	Allentown, Pa.	D. J. Lonergan
Lehigh Structural Steel Company	Allentown, Pa.	C. L. Kreidler
	Allentown, Pa.	R. W. Marshall
Lehigh University	Bethlehem, Pa.	L. S. Beedle
	Bethlehem, Pa.	Dr. Konrad Basler
Levinson Steel Company, The	Pittsburgh, Pa.	J. B. Levinson
	Pittsburgh, Pa.	Budd Overfield
Lincoln Steel Corporation	Lincoln, Neb.	L. P. Lichtenberg
Louisville Bridge & Iron Company	Louisville, Ky.	C. P. Watson
Lowell, Murphy & Company	Denver, Colo.	Byron Lopp
Lyons Iron Works, Inc.	Manchester, N. H.	Cromer Y. Worrell
	Manchester, N. H.	Robert H. Worrell
McKinney & Son, Inc., James	Albany, N. Y.	Maurice J. Roses
McMannus Steel Construction Co., Inc., R. S.	Buffalo, N. Y.	Wilson H. Pratt
Macober, Inc.	Canton, Ohio	J. W. Hubler
Mahon Company, The R. C.	Detroit, Mich.	H. P. Carichner
	Detroit, Mich.	A. E. Lofquist
	Detroit, Mich.	H. E. Turner
	Detroit, Mich.	W. E. Willard
Marquette University	Milwaukee, Wis.	O. Neil Olson
Martin Company	Denver, Colo.	M. Zuckerman
Meurer-Serafini-Meurer	Denver, Colo.	Theron V. Garel
Midwest Industry Magazine	Topeka, Kansas	K. L. Kennedy
Midwest Steel & Iron Works Co., The	Denver, Colo.	W. E. Bower
	Denver, Colo.	R. R. Canada
	Denver, Colo.	Robert C. Christy
	Denver, Colo.	F. G. Fish
	Denver, Colo.	Paul J. Foehl
	Denver, Colo.	W. E. Geer
	Denver, Colo.	C. E. Merhll
	Denver, Colo.	R. E. Schuckmann
	Denver, Colo.	B. M. Smith
	Denver, Colo.	Maynard L. Troster
Midwest Steel Works, Ltd.	Lincoln, Neb.	Raymond P. Zink
Minnesota, University of	Minneapolis, Minn.	Dr. Paul Andersen
	Minneapolis, Minn.	Dr. Walter T. Graves
Mississippi Valley Structural Steel Company	Chattanooga, Tenn.	Kenneth Cook
	Decatur, Ill.	J. Parke Boyer
	Decatur, Ill.	Robert J. Ward
	Melrose Park, Ill.	R. T. Hetherlin
	Melrose Park, Ill.	R. C. Kidd
	St. Louis, Mo.	Norman Cohn
	St. Louis, Mo.	N. J. Law

REGISTRATION LIST

COMPANY	CITY	INDIVIDUAL
Missouri Valley Steel, Inc.	Leavenworth, Kansas	George Huvendick
	Leavenworth, Kansas	William I. Oliver
Morris Wheeler & Co., Inc.	Philadelphia, Pa.	Wm. L. Kern
Mosher Steel Company	Houston, Texas	J. N. Meyer
	Houston, Texas	W. L. Perry
Muchow, W. C. Assoc.	Denver, Colo.	F. C. Zancanella
Nebraska, University of	Lincoln, Neb.	Prof. A. R. Riveland
	Lincoln, Neb.	Prof. Gerald Swihart
Nedell, Robert S.	Denver, Colo.	R. Nedell
New Mexico, University of	Albuquerque, N. M.	E. Zwoyer
Northwestern University	Evanston, Ill.	John F. Ely
Northwestern Steel & Wire	Sterling, Ill.	J. W. Frasor
	Sterling, Ill.	L. Hillard
Omaha Steel Works	Omaha, Neb.	Frank X. Blechinger
	Omaha, Neb.	Robert H. Reid
Pacific Car & Foundry Company	Seattle, Wash.	Robert LeBlanc
Patterson Steel Company	Tulsa, Okla.	J. A. Montgomery
Paxton & Vierling Steel Co.	Omaha, Neb.	James H. Edwardson
	Omaha, Neb.	Ed Quinn
Peden Steel Company	Raleigh, N. C.	D. E. Schreffler
Phoenix Bridge Company	Phoenixville, Pa.	H. W. Brinkman
Phillips-Carter-Osborn	Denver, Colo.	F. Burch
	Denver, Colo.	D. S. Rand
Pittsburgh Bridge & Iron Works	Pittsburgh, Pa.	H. R. Cameron
	Pittsburgh, Pa.	Robert L. Haenel
Pittsburgh-Des Moines Steel Company	Des Moines, Iowa	D. E. Ginn
	Des Moines, Iowa	George W. Hall
Providence Steel & Iron Company	Providence, R. I.	William W. McLeod
	Providence, R. I.	Hermon L. Toof
Public Service Company of Colorado	Denver, Colo.	H. Hight
Purdue University	Lafayette, Ind.	Prof. J. M. Hayes
	Lafayette, Ind.	S. P. Maggard
Robberson Steel Company	Oklahoma City, Okla.	Gerald B. Emerson
	Oklahoma City, Okla.	Jack Hamilton
Russell, Burdsall & Ward Bolt & Nut Company	Denver, Colo.	H. E. Fryer
	Denver, Colo.	D. A. Garrison
	Portchester, N. Y.	F. E. Graves
Ryan, A. J. Co. & Assoc.	Denver, Colo.	D. E. Fleming
Ryerson & Son, Inc., Joseph T.	Chicago, Ill.	J. A. Munro
St. Joseph Structural Steel Company	St. Joseph, Mo.	R. B. Chesney
	St. Joseph, Mo.	Louis Walter
St. Paul Structural Steel Co.	St. Paul, Minn.	Robert R. Clemens
	St. Paul, Minn.	John H. Comfort
Sargent and Lundy	Chicago, Ill.	Max Zar
Schrader Iron Works, Inc.	San Francisco, Calif.	O. W. Schrader
Sheffield Division, Armco Steel Corp.	Denver, Colo.	W. P. Brock
	Kansas City, Mo.	L. M. Alexander
Smith & Caffrey Steel Co., The	Syracuse, N. Y.	A. M. D. Cassel
Southern Engineering Company	Charlotte, N. C.	Leslie G. Berry
	Charlotte, N. C.	Hollis L. Hance, Jr.
Southern Methodist University	Dallas, Texas	E. E. Walters
Stanford University	Stanford, Calif.	Prof. James M. Gere
Star Iron & Steel Co.	Tacoma, Wash.	Edward N. Allen
Stearns-Roger Mfg. Co.	Denver, Colo.	T. F. Buirgy
	Denver, Colo.	W. L. Haney

COMPANY	CITY	INDIVIDUAL
Stearns-Roger Mfg. Co. (Cont'd)	Denver, Colo.	H. W. Hempel
	Denver, Colo.	P. W. Mason
	Denver, Colo.	R. J. A. Thomson
	Denver, Colo.	E. T. Watkins
Sterling Steel & Supply Company	Denver, Colo.	Joe Burke, Jr.
	Denver, Colo.	Jack A. Speyer
Stupp Bros. Bridge & Iron Company	St. Louis, Mo.	Bob Stupp
	St. Louis, Mo.	Neil Stueck
	St. Louis, Mo.	R. C. Zahniser
Sudler, James Assoc.	Denver, Colo.	Bob Burger
Taylor & Gaskin, Inc.	Detroit, Mich.	Carl A. Daniel
Texas A. & M. College	College Station, Texas	Henson K. Stephenson
U. S. Bureau of Public Roads	Denver, Colo.	A. J. Siccardi
	Denver, Colo.	King Burghardt
	Denver, Colo.	Robert E. Frost
U. S. Bureau of Reclamation	Denver, Colo.	Robert Sailer
	Denver, Colo.	Harvey C. Olander
U. S. Steel Corp.	Dearborn, Mich.	Jack A. King
	Pittsburgh, Pa.	C. G. Schilling
	Pittsburgh, Pa.	J. A. Gilligan
	Pittsburgh, Pa.	J. R. Hamilton
	San Francisco, Calif.	R. L. McKillip
U. S. Steel Supply Co.	Pittsburgh, Pa.	Carl E. Seidl
United Structural Steel Co.	Worcester, Mass.	Harry W. Kumpey
Vermont Structural Steel Corp.	Burlington, Vt.	Robert D. Paterson
	Burlington, Vt.	Paul K. Viens
Vierling Steel Works	Chicago, Ill.	Lewis MacDonald
	Chicago, Ill.	Richard W. Molthop
Vincennes Steel Division Novo Industrial Corp.	Vincennes, Ind.	Frank A. Becher
Vinnell Steel	Irwindale, Calif.	H. S. Dewdney
Volunteer Structures, Incorporated	Nashville, Tenn.	E. M. Dougherty
Vulcan Manufacturing Company	Fond-du-lac, Wis.	Paul K. Kottke
	Fond-du-lac, Wis.	Robert E. Roughen
W & W Steel Company	Oklahoma City, Okla.	Tommy McGill
	Oklahoma City, Okla.	T. W. Winneberger
Weaver & Co., Inc., Frank M.	Lansdale, Pa.	Roger W. Derby
Weirton Steel Company	Weirton, W. Va.	E. A. Schiele
West End Iron Works	Cambridge, Mass.	Sol Horwitz
Western Detailing Company	Denver, Colo.	E. C. Shepard
	Denver, Colo.	L. R. Summerhayes
	Denver, Colo.	C. L. Treber
Western Steel Company	Salt Lake City, Utah	William L. Kirk
White, Ken R., Consulting Eng.	Denver, Colo.	L. H. Chesebro
	Denver, Colo.	J. W. Osnes
	Denver, Colo.	Safarian Sargis Sako
	Denver, Colo.	T. V. Stradly
Whitehead & Kales Company	Detroit, Mich.	L. J. Knapp
	Detroit, Mich.	J. E. Mueller
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