SPEAKERS' TABLE AT WEDNESDAY OPENING SESSION

Shown (left to right): Mr. T. R. Mullen, Chairman; Capt. Emil H. Praeger, Messrs. Walter H. Weiskopf, T. R. Higgins and Jack Singleton

THE INSTITUTE'S ENGINEERING STAFF

Front — Messrs. Bell, Stetina, Higgins, Singleton, L. Abbett Post (Executive Vice President) and Miller
INFORMAL GATHERINGS
PRIOR TO WEDNESDAY EVENING DINNER

Front — Messrs. Calvin Loughridge, T. U. McAllister and H. E. Perry
Rear — Messrs. R. B. Reilly, T. R. Higgins and W. L. Perry

THE WEDNESDAY MORNING SESSION
CONTENTS

Wednesday Morning Session, April 23, 1952

Presiding Officer: T. R. MULLEN, President, Lehigh Structural Steel Co., New York, N. Y.

"DESIGN AND CONSTRUCTION OF UNITED NATIONS GENERAL ASSEMBLY BUILDING" —

"LEVER HOUSE — A LIGHTLY LOADED TWENTY-ONE STORY STEEL FRAME" —
MR. WALTER H. WEISKOPF, Weiskopf and Pickworth, Consulting Engineers, New York, N. Y. .... 10

DISCUSSION: ......................................................................................................................... 14

Wednesday Afternoon Session, April 23, 1952

Presiding Officer: JONATHAN JONES, Chief Engineer, Fabricated Steel Construction,
Bethlehem Steel Company, Bethlehem, Pa.

"THE INSTITUTE'S RESEARCH PROGRAM" — A Panel Discussion
PART I — MR. J. O. JACKSON, Pittsburgh-Des Moines Steel Company, Pittsburgh, Pa.,
Chairman, A.I.S.C. Committee on Steel Structures Research .............................................. 16
PART II — MR. LYNN BEEDLE, Assistant to the Director, Fritz Engineering Laboratory,
Lehigh University .................................................................................................................. 21
PART III — DR. NATHAN M. NEWMARK, Professor Structural Engineering,
University of Illinois ............................................................................................................ 24

DISCUSSION: ......................................................................................................................... 29

Thursday Morning Session, April 24, 1952

Presiding Officer: ROBERT J. WOOD, Chief Engineer, Mississippi Valley
Structural Steel Company, Decatur, Ill.

"PROBLEMS OF WELDED FABRICATION" — A Panel Discussion
PART I — MR. BOYD S. MYERS, Robert J. Cummins, Consulting Engineers, Houston, Texas .. 33
PART II — MR. W. L. PERRY, Mosher Steel Company, Houston, Texas ............................... 38
PART III — MR. J. R. STITT, The R. C. Mahon Company, Detroit, Michigan ....................... 42

DISCUSSION: ......................................................................................................................... 46

Thursday Afternoon Session, April 24, 1952

Presiding Officer: T. R. HIGGINS, Director of Engineering, A.I.S.C.

INFORMAL DISCUSSION ON CURRENT ENGINEERING TOPICS ........................................ 51

REGISTRATION LIST ............................................................................................................ 60
The National Engineering Conference of the American Institute of Steel Construction convened at ten o'clock at the Hotel Commodore, New York, New York, Mr. T. R. Mullen presiding.

CHAIRMAN MULLEN: I have been asked by President Wood to preside at this morning's meeting. I feel that I would like first of all to read a wire from President Wood who is in Europe. He said:

PLEASE EXTEND MY BEST WISHES FOR SUCCESSFUL MEETING FOR ALL THOSE PRESENT AT ENGINEERING CONFERENCE.

Gentlemen, this is the fourth of the series of conferences of engineers sponsored by the American Institute of Steel Construction, and I feel it is a great privilege to welcome you to this meeting, which I hope will be a continued policy of the Institute for many years. You know, I served two years as President of the Institute and during that time I threw all the support and weight that I could to the engineering facilities of the Institute, because, after all, without the engineering force that we use to promote our industry, we are as nothing. Our engineering staff is an excellent one, and I think does great credit to us.

Gentlemen, our first speaker this morning is a man who has had varied experience in very important work. First of all, he is one of the partners of the eminent engineering firm of Madigan-Hyland. He was born in New York, graduated from and later became head of the Civil Engineering Department of Rensselaer Polytechnic up in Troy for a number of years. He is a Captain in the United States Navy, Civil Engineer Corps. He received the award of Legion of Merit for work done for the Bureau of Yards and Docks during World War II. He is a consulting engineer on the United Nations' building, and on the renovation of the White House. He has also been chief engineer on a number of parkways in and around New York.

Gentlemen, I take great pleasure in introducing to you Captain Emil H. Praeger.

Design and Construction of United Nations General Assembly Building
CAPTAIN EMIL H. PRAEGER

Mr. Mullen, and members of the American Institute of Steel Construction: I am very happy to be with you this morning. I didn’t notice until I glanced over the program last evening that the subject of my paper is confined to the "Design and Construction of United Nations General Assembly Building." In preparing some notes, I made my talk a little more general and covered the meeting hall buildings as well. If you are not interested in the meeting hall and the other buildings, just consider my comments as introductory remarks to the general assembly.

The United Nations Headquarters is composed of a group of interconnected buildings located adjacent to the East River between 42nd and 48th Streets east of First Avenue in New York City. The site was formerly occupied by slaughter houses and run down industrial buildings, and the adjoining area contained old dwellings, stores, shops, etcetera.

After the War a real estate firm purchased the property and plans were under preparation for constructing a development of modern apartment buildings with stores, a theatre and similar structures.

In 1946 and 1947 a committee consisting of delegates of the United Nations was searching for a site to construct a permanent home to take the place of the temporary quarters then located in Flushing Meadows Park. This site (Flushing Meadows), was offered to the United Nations by the City of New York, without cost, but was rejected. Some members of the Committee favored a suburban site, of several hundred acres, preferably in Westchester County, so that they could build a "city" complete with apartment houses for employees, homes for delegates, amusement center, library, stores and other facilities. Several other cities offered sites and most of these were visited by the site selection committee.

The survey for sites dragged and finally a dead line was set for the selection. Officials of New York City and other prominent people were most anxious that this city
tion to English, the languages used are French, Russian, Chinese and Spanish. By turning a switch at the side of each seat one may select the language desired.

There are other large rooms below the main floor, and the basement contains part of the large parking garage. The upper floors contain lounge rooms, offices and miscellaneous utility rooms.

In plan, the side walls of the building are concave, the roof dips from the north to the center and then slopes up again to the south end. Transversely the roof curves from the center to the sides so that a complicated warped surface results. Over the main hall is a dome surmounting a cone, the axes of both of which tilt to the north at an angle of 5 degrees with the vertical, resulting in the only tilted dome that I know of. One reason for tilting the dome was to obtain greater height over the podium, or speakers' platform, and less in the rear of the room over the relatively less important area assigned to the public.

The inner walls of this room are cone shaped, extending from the base of the dome down, but not to the floor. Behind this wall, which is of corrugated hardwood hung from sloping steel members, are the booths for interpreters, photographers and various other technicians.

The north wall is entirely of glass, measuring 216 feet wide and 71 feet high. The glass panels are supported on steel muntin bars and mullions which in turn are supported on steel box shaped pilaster columns.

The south wall, also of glass, sets back 28 feet from the south building line and the roof cantilevers out over this space.

The east and west exterior walls are of English limestone and marble, unbroken by door or window openings except at the lowest level.

The dominant structural feature of this building is the cantilever. Cantilever stair landings, cantilever booth construction, cantilever theatre type balconies, cantilever construction at the south end of the roof and finally, cantilever construction to support the sloping cone and dome.

True there are steel arch rib stairways, offset columns, extremely heavy built-up girders, one at the roof level weighing about 150 tons, the usual swiss cheese webs of rolled members for ducts, many complicated framing details such as the podium pit, the lack of straight continuous lines in the framing pattern, the cone and dome and the incident bracing, but among the most difficult problems, from an erection point of view, were the cantilevers and the cambers provided for dead load deflection.

I will try to describe some of the problems with which we were confronted in designing the dome and with which the erecter had to cope in erecting it.

The dome is composed of 24 ribs curved to a radius of 73 ft. 1 in. and thrusting outward against a tension ring made of a series of bent 24 WF sections spliced together. The tops of the ribs are attached to a box girder compression ring. The dome is supported on a strut at each panel point which is radial to the center of the dome and cone and lies in the plane of the cone supporting the dome.

The base of the dome is 98 feet in diameter and the base of the supporting cone is approximately 104 feet in diameter. A rectangular pattern of principal members support the cone which in turn supports the dome. To the north there is a transverse girder, 132 feet long located 56 feet from the center of the dome. To the south, a transverse girder of 80 foot span is located 56 feet from the dome center. Our column spacing pattern was generally 28 feet by 26 ft. 6 in. To the east and west, there are longitudinal members of 28 foot span located 67 feet from the dome center. From this rectangular supporting frame cantilever members extend out to and support the base of the cone.

On either side of the N-S center line there are a total of 12 cantilevers, each of a different length. The base of the dome and therefore the base of the cone had to be erected at the designed elevations to produce a plane surface under total dead load conditions. As each cantilever is of a different length the magnitude of deflection of each is different under full dead load. As these cantilever members are in turn supported on cambered members of varying spans and from points at varying distances from the center of the supporting members, the deflection problem became quite complicated.

Dead load deflections and required dead load cambers were computed for all members, both cantilevers and fulcrums.

The cone and dome, however, could not be riveted until the supporting members were at the final dead load elevations. These elevations are normally produced by the weight of the superimposed materials but these could not be installed until after the riveting was completed.

Several erection schemes to solve the above problem were studied. One was to pull the cantilever tips down to final position by means of cables and to then make the connections of X-bracing between supports, rivet up the entire dome, apply the dead loads and then cut the cables and hope that, all calculations being correct, the dome would be freely floating in a position of equilibrium.

Incidentally, these problems largely fell on our lap as designers, rather than on the erectors as is the more usual case, because of the very complicated nature of this problem and the fact that we had all the data.

There were several practical difficulties in using this scheme:

1. To pull the points of dome support fully down to final position would have required cable loads so
2. In computing deflections it is difficult to know the exact effective moment of inertia of variable irregular and built-up members.

3. It had to be kept in mind that at the time of applying the cable loads some members might already be fire-proofed and the effective moment of inertia had to be estimated.

4. After pulling the tips of cantilevers down to a precomputed elevation, how could we assure ourselves that the structure was behaving as computed and that the cantilever in question was taking its proper share of the load.

Calculations indicated that if the cantilevers were pulled down to a position 3/8" above final position, it would bring the cable loads down to workable values.

The dome and cone being a rigid space structure could settle the remaining distance gradually without suffering undue strain. Slight inequalities in the support reactions, though undesirable, were provided for in designing the X-bracing of the space frame cone structure.

The proper behavior of the structure depended to a large extent on the checking of deflections under progressive loading and making adjustments in camber where necessary when changes in field conditions changed the problem constants.

The final positions and unloaded positions having been calculated, it was then necessary to specify the cantilever tip elevation at erection. Cantilevers were adjusted to these elevations by means of cables running over strongbacks and then the strap plate holes were field reamed and riveted.

The contractor's progress could not be interfered with or dictated. If some concrete dead load were placed, or if a derrick were moved near the fulcrum steel, the new elevation of cantilever tip had to be calculated and corrections made in time to assure riveting the cantilever with the proper camber. A mistake at this point would have been rather difficult to correct.

After riveting all cantilevers the cable load required to pull down to a predetermined elevation was computed. In the case of the 132 foot fulcrum girder carrying 8 cantilevers, it was necessary to solve 8 simultaneous equations to find these loads.

Near the points where the cables were tied down at their lowest points, flat steel bars connected by shackles to the cable and then tied down were used for the application of electrical strain gauges to measure the strain in the plates. Combining these strain measurements with the average E value for the steel found in laboratory calibration, we were able to determine the actual cable loads to a high degree of precision (i.e., within 100 pounds of the actual load).

The cables were tightened in groups until the strain gauge operator reported that the precomputed cable loads had been reached. At this point record was made of the cantilever tip elevation. The average error between actual tip elevation and computed elevation under load was about 1/4" to 3/8". Shims were then placed on top of the cantilever tips to correct these errors. Again during this part of the work, changes in contractor's procedure and movement of dead loads required recomputation of cable loads immediately to meet the new loading conditions.

It was interesting to note that after the cable loads had been on for some time, changes of temperature caused changes in cable length, load, and consequently elevation of cantilever tip. However, when the cantilever tips were pulled to the original elevation or when the change in temperature duplicated the original conditions, the cable loads measured the same as originally, proving that no measurable slippage of rivets in the reamed strap plate holes had taken place.

The cables were left in place until all the dead load of the roof and dome had been placed. Then they became fairly slack and were removed.

Here are some general remarks:

Holes in webs of rolled members for the passage of ducts and pipe are the rule rather than the exception. While the New York City Building Department has no jurisdiction over these buildings, the requirements of the New York City Building Code were followed in general. Rivets, and bolts, and some incidental welding were used. Steel to be incased in concrete had a coat of shop paint, while other steel had an additional coat of field paint. Shallow members were fireproofed with concrete, while deep girders and trusses were protected by metal lath and vermiculite plaster.

Our design drawings were carefully prepared and all dimensions both horizontal and vertical (levels) were computed to thousandths of a foot or about 1/60 inch. The preparation and checking of shop drawings was a major task, and required close cooperation between our office and the fabricator and erector. I can report that there were no fabrication or erection errors of any consequence and was pleased to receive a letter from the steel contractor, after completion of erection, complimenting our office on the lack of disagreements on the entire project which extended over a period of almost three years.

Now, I mentioned the names of some of the people connected with it. You know the American Bridge Company had the contract. Mr. Vanderbick, Mr. Fine, Mr. Graham and many others right up to Dr. Webb, too, took a great interest in this building. Those connected with the structure, representing the United Nations principally, were Mr. Wallace Harrison, Mr. James Dawson and in our office particularly on the General Assembly Building, Mr. Philip Levitan did a very good job.
Now I have a few slides that illustrate some of these features that I have told you about.

Figure 1 pictures the nearly completed south facade of the Meeting Hall. Note the cantilever construction, above. This photograph shows the glass wall, the cantilevered walkway, and the stairs down to the promenade that extends over Roosevelt Drive. The promenade has cantilevers 38 feet in length and forms a roof over Roosevelt Drive.

Figure 2 illustrates the erection of a cantilever and moles that support stringers for an interior stairway near the north facade of the General Assembly. The spaces between the long unsupported box mullion columns at the north facade, also shown in the picture, will be filled in with glass to form the north wall of the structure.

Figure 3 gives an overall view of the General Assembly Building looking north. Note the finished glass face of the west wall of the Secretariat Building at the right of the picture. Beyond that may be seen what we call the 'neck,' connecting the General Assembly Building and the Meeting Hall, hidden in this view by the Secretariat. Now, looking at the steel framework of the General Assembly, notice the swayback shape of the roof in the north-south direction, and the convex crown of the roof in the east-west direction. The roof is high at the north and south and dips in the center. It is surmounted by the sloping dome and cone about midway in its length. Also note the concave west wall at the left side of the picture. The elevation and location for all working points on the roof of this structure were computed to four decimal places.

Figure 4 is a similar view looking south-west. The long unsupported box columns, that form the north facade and support the roof of the structure, are clearly shown at the right. Here again you get an idea of the shape of the roof, and of the concave east wall.

Figure 5. Here is a picture of the General Assembly Hall taken from a position looking north-east. The location and tilt of the dome and cone, and the profile of the roof are clearly shown. As I tried to describe it, the supporting framework under the cone and dome is in the shape roughly of a hollow rectangle much larger than the base of the cone at the roof line. Within this rectangle, framed by the large girders on the north and south and by other framing on the east and west, project out a number of cantilever members that support the dome.

Figure 6 is just a general view looking up into the dome. The temporary false-work tower used during erection is still in place below the small ring girder that joins the dome ribs. One of the cables, with a turn-buckle and clevises, used to pull down the tips of a cantilever can be seen in the foreground.

Figure 7 gives a close-up of the dome and cone as seen from its east side. As in the previous photograph, the temporary falsework tower is still in position underneath the center of the dome. Just beyond the far side of the cone can be seen a portion of the heavy transverse girder that supports some of the cantilevers that support the cone and dome. The tilt of the dome, and the warped surface of the roof framing is clearly shown.

I think that is all. I appreciate the opportunity of speaking to you today.

CHAIRMAN MULLEN: Thank you very much indeed, Captain Praeger.

We have another guest speaker who is a partner of the firm of Weiskopf & Pickworth, located here in New York. Mr. Walter Weiskopf was educated at Rensselaer Polytechnic Institute as was Capt'n. Praeger. He started with the American Bridge Company, and later became New York representative for Carnegie Illinois Steel Company, and occupied the same position that his father had held 40 years before. That is an interesting thing in his professional biography.

He joined Weiskopf & Pickworth in 1926, and traveled to South America on several engineering projects. He is the author of many technical articles, and specializes in rigid frames. During World War II, he was for three years civilian head of construction in the New York District for the United States Army. He was the engineer for the buildings occupying three corners at Wall and Broad Streets. No. 1 Wall is one of those buildings, I believe. The fourth corner building, of course, was erected a little before his time—it is the United States Treasury building. He was engineer on the building of the Hayden Planetarium for which our company had the privilege of supplying and erecting the steel.

He is an inventor and holds many patents dealing with engineering, and of all things he holds a patent on a steel plate girder.

I take pleasure in introducing to you Mr. Walter H. Weiskopf.

Lever House—A Lightly Loaded Twenty-one Story Steel Frame

MR. WALTER H. WEISKOPF

Mr. Mullen, gentlemen: I hope you will be interested in what I have to say about the Lever House Building. I have been rather interested in it, not so much because it is another building, but because I think it is rather
Below: view of both units in operation.

Above: plan of carriage and platform. Shaded portion indicates elements of the platform.
typical of some of the trends which are going on in the modern buildings. I am thinking particularly of the lightweight floors and lightweight walls which it uses, and the effects of the lightweight construction on the design of the steel frame.

Let us talk about the walls first. Traditionally walls are very thick, very heavy masonry things. That, of course, comes from the history of buildings in the old days. There were no columns and the vertical elements were old-fashioned masonry bearing walls.

I recall that my father used to tell me that around the turn of the century he had quite an argument with the authorities to allow him to use steel columns to carry the walls. Finally, the New York City authorities made a compromise and allowed him to put in the steel columns, provided that the masonry walls were made just as thick as if no columns were there. Eventually, however, the argument was won and steel frame buildings came into being.

For a good many years, however, the New York City Code, and codes generally, I believe, throughout the country specified that all paneled walls, as they were called, must have a 4-hour fire rating and a minimum thickness of 12 inches. There was one peculiarity about the code. It always seemed peculiar to me. Traditional exterior walls of buildings may have windows. The windows had no fire rating whatsoever, and there was never any stipulation in the code as to what percentage of the exterior walls could be window openings. The designer, therefore, was allowed to make the window openings as large as he wished, sometimes perhaps 80 per cent or more of the total area, but the remaining 20 per cent had to be 12 inches thick and had to have a 4-hour fire rating.

As time went on, architects and builders became conscious of the fact that 4-hour exterior walls were rather unnecessary on high buildings. It is hard to conceive of a fire of an intensity of very high temperature lasting for 4-hours outside of a wall on the 30th story of a building.

There was a great deal of discussion on this point and only a few years ago, in 1948, the New York City Code was changed. The fire rating for exterior walls was changed from 4-hours to 2-hours. The minimum thickness requirement was eliminated, and instead the code provided that all exterior walls should withstand a wind pressure of 30 pounds a square foot. This was obviously a tremendous improvement. Some of the architects still feel that it is conservative and work on securing adoption of thinner walls. However at the present time that is the law in New York City and that is the law under which the Lever House building was constructed.

Bear in mind that the 2-hour rating can be accomplished with about four inches of masonry, but you cannot build a wall on a panel, say, 28 feet between columns and 12 feet from story to story, four inches thick that will withstand a 30 pound wind. It was, therefore, necessary in the Lever Building to provide for vertical mullions to withstand the wind pressure, support the windows and give lateral stability to the enclosure wall. So much for the wall for the moment.

The other trend in lightweight construction is in the floors. The floors have traditionally been reinforced concrete. In the old days, other types were used such as terra cotta arches. They are not used any more.

A 3-hour rating is needed in New York City, and, I believe, in most cities. On top of the concrete floor, it is usually necessary to place what is called a "fill", a lean grade of concrete in which the electrical conduits are buried and on top of which the finished floor is placed. The concrete floor, of course, encases and fireproofs the beams.

A number of years ago, experiments were made with vermiculite plaster as fireproofing, and at the present time in New York City vermiculite plaster is approved for fireproofing of structural steel. It is given a 3-hour rating.

It has therefore become possible to use a cellular steel plate floor on top of the steel beams, a 2-inch concrete slab on top of the cellular plate floor, and a vermiculite plaster ceiling under the steel beams in most modern first-class fireproof buildings. There is nothing corresponding to the "fill" I mentioned to cover the electrical conduits, because the electrical work can be placed in the cells of the cellular steel plate floor.

It is obviously very much lighter in weight as compared to the old-fashioned masonry construction.

Vermiculate plaster can also be used to fireproof columns. In this instance, one-inch plaster on metal lath is placed on furring channel around the column, and in order to get a 4-hour rating in New York City, the space between the column and plaster had to be filled with loose vermiculite.

There were several effects on the structural steel design that resulted from using this rather lightweight construction. It is obvious immediately that the ratio of live load to dead load in the floor construction is changed. The live load is the same but the floor construction itself is much lighter. It is also noticed that the beams are not encased in stone concrete. For both of these reasons the deflections of the beams under the vertical loads have to receive more consideration than in the old-fashioned designs. The stiffening of the beams due to the encasement of stone concrete is very considerable.

Similarly, for exactly the same two reasons, the deflections from wind have to be studied more carefully. Since the building itself is lighter, the dead loads are smaller, all the members are smaller, and the wind stresses are a large proportion of the total stress. For this reason, and
because the members generally, both columns and beams, are not encased in stone concrete haunches the wind deflections are larger than in a building designed to use the old-fashioned materials.

There is one other peculiarity which we ran into on the Lever building. The tower part is fairly narrow in one direction and had to be designed to withstand the wind forces. In the other direction it is quite long, and if the building had been an old-fashioned one, no consideration would have been given to the wind in this strong direction. However, here you have a building with extremely light walls, with very light, framed shallow beams, spanning the long way. For these reasons we felt it was necessary to provide wind bracing in both directions.

I think that a few slides will illustrate these points more easily than I can describe them.

Figure 1 gives you an idea of the general type of structure. It is on Park Avenue just a few blocks north of here. It occupies the entire area from 53rd to 54th Street up to the third floor. Above that point, it is a fairly narrow tower going to about 21 stories plus a considerable amount of penthouse structure. This is a view looking northwest at the Park Avenue front.

I might mention here, as you will notice, the columns are not on the building line on Park Avenue. The front of the Building cantilevers about 10 feet from the line of columns. Originally, that was done at the request of the architects. They did not want to show columns in front. They asked us whether it could be done feasibly. We studied it and found it could be done without any great difficulties, but it had a great advantage from our angle. The tunnel of the New York Central Railroad is just below Park Avenue, and the tracks are beginning to widen out at this point to go into the Grand Central Station. The New York Central Railroad occupies the full space, from building line to building line, underneath the road and sidewalk so that the tunnel was directly on our building line. Moving the front line of columns 10 feet back simplified our foundation problems tremendously. As a matter of fact if we had not had the cantilever on every floor, we would have had to put a large cantilever in the basement to find an area away from the New York Central tunnel.

Figure 2 is a view from 53rd Street. It shows the cantilevers and the length of the tower part of the building in the east-west direction.

These illustrations are largely to give you a general idea of the building's appearance, and I don't think it is necessary to take too long on them individually.

Figure 3 is a similar view from 54th Street.

Figure 4 shows the construction further along. The wall construction was starting up. The vertical mullions were not part of the contract. They were furnished by the General Bronze Company who did the window framing. Nevertheless, they are used as the support for the walls and the sash. They have some light structural framing in them even though they are not structural members.

Figure 5. This typical section of the spandrel beam, exterior wall, and floor construction gives an idea of how both were built. The entire wall thickness is four inches of solid cinder block. That is all that is needed for 2-hour fire rating. Behind the wall is a two inch thickness of cellular-glass insulation.

The architectural design was such that glass appears not only at the windows but lined up with it there is glass past the spandrels, so the effect on the outside is an all glass building. Viewed from the outside, the glass at the spandrels looks darker because it has construction behind it. All of the glass is greenish-bluish color because it is heat resistant. The effect achieved is to produce horizontal lines of two colors of green glass. Vertically you have some very light stainless steel colored mullions.

You can appreciate from this how much lighter the wall is than the old-fashioned 12-inch brick wall with just a few openings in it for windows. The structural steel spandrel beam is quite a distance back of the wall. The reason is to accommodate the air conditioning systems.

On top of the structural steel beams is a cellular steel floor, in this case Robertson "Q" floor, and on top of that two and one-half inches of stone concrete and then asphalt tile. On the underside of the structural steel frame is a one inch vermiculite plaster ceiling. The two inches of concrete above the steel floor and the vermiculite plaster ceiling under the structural steel furnish the entire fireproofing for floor construction.

The ducts for the air conditioning in the building are below the fireproofing ceiling. The building is entirely air conditioned and there is a very extensive duct system. Below the ducts is the architectural ceiling. There are two ceilings in the building. The upper one is for fireproofing; the lower for architectural effect. There is recessed lighting in the space and a very complicated system of ducts.

Study was given to eliminating one of the ceilings, and using one ceiling only. There were many difficulties. The fire rating on the structure on the basis of ducts between the structure and the fire resistant ceiling was questionable. It would have required fire dampers in all the ducts at frequent intervals. It would have required fire cut-off panels in the ceiling. Finally, it got to be so complicated and so expensive that it was decided to eliminate the question and to put in two ceilings, one for fireproofing and one for hiding the ducts and recessed lighting.

I would like to say that the decision to use "Q" flooring and a light steel frame was not made lightly in this
building. It wasn't done simply because the architects and engineers thought they would like to build a real modern or unusual structure. In the early stages, several ways of building it were studied, and plans for typical panels up and down the building were drawn up for the conventional construction—steel beams, a 4-inch lightweight concrete slab, and a 4-inch fill or finish. Plans were also drawn up for similar panels showing a lightweight construction. Estimates were made. The estimates had to be made by ourselves and the architects jointly, because the estimates affected a great many materials other than just the structural frame. The estimates had to include not only the steel work, the two ceilings, and the concrete on top in the case of lightweight design, but had to include also the floor fill and what was very important the electrical conduit system on a conventional design.

On this design the electrical wires go through the cells. On the conventional type, conduits had to be placed on top of slabs in the floor fill. It was found that, if the owner required a close spacing of electrical conduit system, the lightweight construction using the "Q" floor was less expensive. On the other hand, for a different type of building, if a closely spaced electrical system was not needed, then the "Q" floor was more expensive. The 'Q' flooring itself is not a cheap article. The justification from an economical standpoint in this case came from overall consideration, including an elaborate under-floor electrical duct system.

Figure 6. This is a floor plan of the building that illustrates a couple of things. It is only two bays wide in the north-south direction. This plan is above the third floor in the tower portion. The framing is very simple. Note the 10-foot cantilevers at the east end of the floor. The east-west bays are 28 feet, which is fairly long for office building construction. The small 6-inch channels around the columns are to support the "Q" flooring where the cells are interrupted by column details.

The wind bracing systems are on each bent, in the front and around the elevators in the rear. There are deep knee braces around the elevators. That is in the weak direction of the building. In the strong direction of the building, the wind bracing, which we felt necessary to include, is placed behind the elevator shafts to provide full depth knee braces.

In Figure 7, the details at right show the wind bracing connection used. In order to get sufficient strength at the column, a stub was riveted to the under side of the beam and a split beam "T", top and bottom, was used. That had some advantage other than just to save a little steel.

I might first state that the vermiculite fireproofing ceiling is generally at a level immediately under the girders and all of the beams and girders on the floor are of approximately the same depth, 14 inch beams or 14 inch girders, so the vermiculite plaster can go through at one level. The only break occurs at the column stubs at wind bracing connections. These breaks occur at all of the columns, exterior and interior.

The ducts for air conditioning were placed between the stubs, and the architectural ceiling was placed below the stubs and ducts at a uniform elevation. It is, therefore, possible to get the ducts and the wind bracing connections in the same vertical space. There is a great deal that goes on in that space between the ceiling on one floor and the floor above. It seemed like a great deal to us to use three feet four inches in every floor just for construction, but when you consider what goes on, I think it is well used.

In this case, the structural engineer and the mechanical engineers both used it as you see from this sketch. The diagrams at left show the arrangement of wind bracing towers. In the weak direction on each side of the elevators the knee braces are full depth but leave a space in the middle for elevator lobbies. In the strong direction of the building, they are behind the elevators, so full length knee braces could be put in.

Figure 8. I might give you a little bit of an explanation of this first if you haven't seen it in the newspaper. The building is entirely air conditioned. It is all glass on the sides. The architects and the engineers for air conditioning were very anxious to have all the glass fixed sash. That was for several reasons. In the first place the fixed sash is cheaper, and secondly it doesn't leak as much. The difficulty in using fixed sash is that there is no way to clean it from the outside. So, somebody—I am not sure who; I think it was somebody in the architect's office—conceived the idea of cleaning the exterior of the building, both the glass for the windows and the glass for the spandrels from a scaffold hung from the outside of the building and lowered up and down. It had many advantages.

In the first place, the cost of the scaffold was absorbed by the difference in costs between fixed and ventilated sash and, of course, the cost of operating is less because of the saving in time in climbing in and out of windows. In the second place there is no disturbance to office people, as when window washers come and jump over your desk to get out the windows to perform their work.

After a considerable amount of study on the part of the architects, the Otis Elevator Company, ourselves, and the mechanical engineers, this scheme was evolved. The sketch is a reproduction of one of the architect's drawings. It was necessary to talk to the authorities—that is the City authorities and the State Industrial people—to get permission to use it.

The scaffolding runs up and down the exterior of the building. It runs on a device at its mid-point that is clamped to a stainless steel rail on each column. We did not feel it was safe to use a normal painter's scaffolding.
as a permanent thing. We felt very strongly that it was necessary to provide a positive fastening between the scaffolding and the building. The two ends simply have rollers which press against the building. An electrical hoist moves it up and down.

When it is in upper position, the scaffold is above the parapet and then the entire machine can run horizontally on rails. It runs not only horizontally but it runs around the corner on Park Avenue, and when not in use it runs back on the main portion of the roof at the rear of the building, out of the way.

That gives you largely what I have to say regarding the building itself. If there are any questions that you may wish to ask, I hope I can answer them.

Chairman Mullen: Thank you very much, Mr. Weiskopf. I perhaps was remiss in not offering the audience an opportunity to engage Captain Praeger in questions. I think that if any of you have questions to ask either of our speakers, this is the time to do so.

One question occurred to me when I saw that scaffolding arrangement. I know that window washers are only supposed by union edict to wash a certain number of windows a day. But when they are out on that scaffold, they will either have to work or stand there and loaf.

I had another question that I wanted to ask Mr. Weiskopf. I was curious to know what the construction material is in that lower ceiling, the one below the vermiculite.

Mr. Weiskopf: That is ordinary plaster. It is also acoustical so as to quiet the noises inside. The architect can put in anything he likes as long as he doesn't make it too heavy.

Chairman Mullen: Any other questions, gentlemen?

Mr. Corbit (Los Angeles): I would like to ask about the insulation of the walls at the spandrel section. Do you notice on hot days, bright sunny days, the transmission of heat to the desk space immediately adjacent to the inside of the walls?

Mr. Weiskopf: The insulation provided, as you remember from the slide, is two inches of fiber glass insulation on the inside of the wall. I cannot answer the question as to whether any heat transmission is noticeable. The building is just being occupied now and has not gone through a summer yet.

Mr. Corbit: The reason I ask is that I have seen a building with a similar type of exterior wall. I was in it recently and talked to one of the men who occupied a desk along the outside wall. The heat coming through the glass was absorbed by the wall and really made it quite uncomfortable right at that spot. Of course, the air conditioning took care of other people in the room.

Mr. Weiskopf: Well, I can speak for the air conditioning engineers. They are an excellent concern. I know them very well. I can only state that I imagine they were conscious of the problem. I can also point out that they took care of supplying the cold air right at the glass. We had to move the spandrel beams in so that a supply of cold air could be injected into the building right at the floor line just behind the glass. Perhaps that will take care of it satisfactorily. I hope it will.

Chairman Mullen: Any other questions?

Maybe we should put a question to Dr. Webb as to how many of his draftsmen went insane while they were detailing that job that Captain Praeger described.

Dr. Webb (American Bridge Company): I might say that that is one job that called for very good draftsmen. I think they got cantilevered to death in working out the details. There was good cooperation with Captain Praeger and his office, through our New York office and the Cambridge plant. Our erection department also had their troubles but everything worked along very smoothly.

Chairman Mullen: Well, I think there isn't any question but that everybody in this room takes his hat off to the American Bridge Company for putting that job up. From the outside, it looks like a sweet job, but, brother, when you look at that design and realize the detail — well, our compliments to you!

Mr. John Griffiths: Mr. Weiskopf, you said that "Q" flooring was selected largely because of the electrical availability feature. What alternate floor constructions did you consider before you made the selection?

Mr. Weiskopf: No other cellular steel floor was considered. "Q" floor at that time — I am not sure whether it is true or not today — was the only cellular steel floor approved in New York City. The one made by another manufacturer had not at that time been approved for use with electrical wiring through the cells. The only studies that we compared with "Q" flooring were all masonry construction with floors of concrete. We studied lightweight concrete floors, stone concrete, long span concrete. As far as I know, that is all. The comparison was entirely between the "Q" floor and basement floor.

As long as I am on my feet, I think it is only fair to mention that the Bethlehem Steel Company supplied the steel for the Lever building. They did a very excellent job and we are very much pleased with them.

Chairman Mullen: We appreciate your advertising. If there is nothing further and no one has any question —

Mr. Santoro (Dave Steel Company): I would like to ask Captain Praeger a question. Every so often you hear about buildings where you go up to the top and start building down. I wonder if it had any practical disadvantages in doing so, I mean with workmen and so forth.

Captain Praeger: That was done on the Meeting Halls Building for the reason I mentioned. It cost money
to do that. It was only done for the specific purpose that I mentioned, because the progressive load would cause progressive reduction in camber, and therefore if the masonry had been built up previously to the under side of the member the additional deflection would crack the masonry.

Mr. Santoro: I was wondering if you needed better workmen to go up to the top and start down.

Captain Praeger: I wouldn't say so. We used the same men. There was a lot of grumbling at our requirements that it be done and it probably cost more money. We did pay additional sums because of the disruption mainly to the concrete contractor who had to start at the top and work down which made it more difficult for him.

Chairman Mullen: Generally speaking, we in the steel business don't encourage erecting from the top down. The sky hook hasn't been invented yet.

If there are no further questions, I declare this meeting adjourned.

(The meeting recessed at twelve o'clock.)
The meeting reconvened at two o'clock, Mr. Jonathan Jones presiding.

CHAIRMAN JONATHAN JONES: Gentlemen, we will call the afternoon session to order. The logical chairman for the technical session would be Mr. Higgins our Director of Engineering, but Mr. Higgins called me and asked if I would take over this session because he thought that possibly some of the members were entitled to a vacation from seeing him on the platform. I felt that possibly that applied quite as much to myself as to him, because you have seen a lot of me up here the last few meetings. But the topic for this afternoon is the participation of the Institute in structural research, and I have been tied in with those efforts over quite a few years, having followed what was going on and helping a little here and there to steer it. That implies that I have some confidence in the fact that structural research is worthwhile, which I certainly have. The longer I am in it, the more I realize that there are many things that have yet to be founded, many ideas that have yet to be confirmed, and many projects that are worth undertaking. Not all the projects that we undertake turn out to be gold mines, or can be. Sometimes we don't find the results that we had hoped for. Other times we do. Some of our research projects have become of immediate commercial importance. Others seem to require some years of study before they are coordinated, before we can really say what it is all about, but by and large I have a great deal of faith that the work is worth-while.

It is not by any means being overdone, and the amount of it that we are doing in the fabricating field is not yet competitive with what our competitors are doing.

If I have anything to say personally about any aspect of our structural research, it is logical for me to withhold that until later because doubtless the speakers for the afternoon are going to cover the same sort of things that I might say.

The first speaker this afternoon is the Chairman of the Institute's Committee on Steel Structures Research, and he will give you a competent survey of what has been done and what is hoped for. I can't follow the pattern of this morning's moderator by citing the school at which he was educated, but I can say that wherever Mr. Jackson obtained his education, he learned thoroughness for he is an indefatigable seeker after the fundamental truth of everything he undertakes. He is a very thorough and interested worker in this field of structural research and I am glad to introduce Mr. Jackson of Pittsburgh-Des Moines Steel Company.

Panel Discussion
The Institute's Research Program—Part I
MR. J. O. JACKSON

Gentlemen: I would like to say first that any opinions that I may express are my own and should not be construed as those of either the American Institute of Steel Construction, or any other members of the Research Committee.

The important purposes of the American Institute of Steel Construction as stated by Mr. R. D. Wood in the President's 1951 Annual Address, are:

1. To increase through technical research the fund of engineering knowledge and to promote the use of steel construction.
2. To stimulate through cooperative effort efficiencies and economies in the design, fabrication and erection of structural steel.
3. To disseminate all such information and to assist in its application through a staff of engineers.
4. To promote the growth of the structural steel industry by expanding its possibilities and markets.
5. To collect and publicize pertinent statistical information.
6. To foster the improvement of conditions and business relationships within and without the industry.

The first three of these six purposes are directly concerned with research. For many years your Institute has
actively supported research work and the membership has benefited therefrom. Some examples of successful Institute research are the Battledock Floor, Lightweight Plaster Fireproofing, Stresses in Rigid Frame Knees, Semi-Rigid Joints, High Strength Bolts and Perforated Cover Plates.

The Committee on Steel Structures Research has the following duties:

1. To advise on the various research programs under way and consider suggestions for future projects.
2. To suggest new fields of research.
3. To approve reports and technical papers before their publication.

While the Institute does not maintain a research laboratory, it does cooperate with organizations doing research work and it contributes money to help pay for research work in fields of particular interest to our members. Projects now under way include the following:

1. An investigation to determine the Behavior of Beams, Columns, and Rigid Frame Knees Loaded Well into the Plastic Range. This work is being done at Lehigh University and is co-sponsored by the Navy Bureau of Ships, the Bureau of Yards and Docks, the Office of Naval Research, the American Iron and Steel Institute, and the American Institute of Steel Construction.

2. The Column Research Council was organized for the purpose of conducting research on Steel Columns and other Compression Members. It is supported by contributions from interested associations, government agencies and companies. The projects under way at the present time include: An Investigation of Residual Stresses in Columns in Structural Frames — at Lehigh University, the Buckling of Rigid Joint Structures with special consideration to Columns in Structural Frames — at Cornell University, Initial Eccentricities — at Purdue University, Local Buckling in Built-Up Columns — at Stanford University, Inelastic Instability as it Affects Local Buckling — at Lehigh University, Torsional Instability — at Illinois University, the Lateral Buckling of Beams as influenced by Torsional Instability — at the University of Washington, the Interaction Formula as influenced by Torsional Instability — at Brown University, and the Stability of Bridge Chords without Lateral Bracing — at Pennsylvania State College.

Mr. Lynn Beedle, assistant to the director of the Fritz Engineering Laboratory of Lehigh University, will describe the research work being done by the Column Research Council at Lehigh University.

3. The Research Council for Riveted and Bolted Structural Joints is continuing its work. This Council is made up of the American Institute of Steel Construction, the Association of American Railroads, the Engineering Foundation of the American Society of Civil Engineers, the Industrial Fasteners Institute, the Illinois Division of Highways, and the Public Roads Administration of the United States. Several reports have been issued and as a result the use of high strength bolts is increasing rapidly.

Professor N. M. Newmark, Research Professor of Structural Engineering of the University of Illinois, is going to tell us about the research work on riveted and bolted joints.

4. A research project recently sponsored by the American Institute of Steel Construction is concerned with the surface preparation and painting of steel structures. The original committee appointed by the Institute led to the formation of the Steel Structures Painting Council, which is now being financially supported by our Institute, the Association of American Railroads, the Steel Plate Fabricators Association, the National Association of Corrosion Engineers, the Federation of Paint and Varnish Production Clubs, and the National Paint, Varnish and Lacquer Association. The Steel Structures Painting Council has just completed the preparation of nine specifications covering surface preparation of steel for painting. These specifications are unique in that the contractor can predetermine his cleaning costs irrespective of the surface condition. These specifications are now out for committee ballot and should be available within a few weeks. Preliminary copies can now be obtained from Mr. Higgins or from the Council.

The Welding Research Council of the Engineering Foundation is sponsored by many supporters including the American Institute of Steel Construction. This Council has a Structural Steel Committee which supervises research work at Lehigh University on Welded Continuous Frames and Their Components, Columns, Continuous Beams, Inelastic Instability, Compressive Properties of Rolled Structural Steel, Residual Stress, and Stress Strain Properties of Steel and Connections. The Welding Research Council is also doing work on Resistance Welding, which has applications in our industry. The Pressure Vessel Research Committee of this Council has an extensive program of research, including the Strain Aging of Pressure Vessel Steels, the Transition from Ductile to Brittle Behavior in Pressure Vessel Steels, Structural Steels for Use in Pressure Vessels, the Use of High Strength, Low Alloy Steels in Pressure Vessels, Creep and Stress to Rupture Properties of Pressure Vessels, the Occurrence of Graphitization in Carbon Steel of Stainless Clad Vessels, Corrosion Fatigue and Stress Corrosion of Pressure Vessels, a Comparison of the Effects of Preheating and of Stress Relieving on the Properties of Welded Pressure Vessel Steels, Known Changes of Behavior in Carbon and Low Alloy Steels within the Temperature Range of $-20$ to $-650°F$, the Bauschinger Effect and its Importance in Pressure Vessels, Heat Treatment of Pressure Vessels, Temper Embrittlement in Pressure Vessels, the Effect of Anisotropy on Flow and Fracture of Steels, and a program for Investigating the Effect of External Loadings on Pressure Vessels.
The projects which I have named cover a broad field and solutions to them are needed. It may appear that almost every question that needs answering is now being investigated, but this is not true. Changing conditions bring new problems, and improvements in steel making and in fabricating processes make available new materials and new methods which usually require additional knowledge for their proper utilization.

The advent of the atom bomb has made it necessary for us to review the designs of important structures in order that we may appraise our chance of surviving an atomic attack. The effect of an atom bomb explosion on a steel structure depends principally on its distance from the center of the explosion. A nearby structure will first feel the effect of a shock front propelled from the explosion center. This shock front consists of a gust of high pressure air blowing away from the explosion. At a distance of 1,000 feet from the explosion center, the shock front will arrive with a velocity of about 800 miles per hour. It will have a pressure of about 36 pounds per square inch gauge and a temperature of about 150°F. The wind force of this gust will be about 3 tons per square foot, or about 200 times the force exerted by a 100 mile per hour wind. The high pressure will immediately start to diminish and after only about one-third second it will be atmospheric. A vacuum stage follows immediately, and after a few seconds a maximum vacuum of about one-third of an atmosphere will be reached. The vacuum will then gradually reduce until the pressure again becomes normal. During this interval the structure will be exposed to extremely high temperature radiation which will set fire to any combustible materials such as paint, wood, roofing, etc. The structure will also be subjected to atomic radiation.

An atom bomb of present size bursting in the air will completely destroy most steel structures of present design within a radius of about one-half mile from the ground center of the explosion. At a distance of one mile from the center most steel structures of present design will be damaged to such an extent that they will have to be completely rebuilt. Figure 1 shows an industrial steel frame building at Hiroshima with sawtooth trusses about 5,000 ft. from ground zero.

At a distance of approximately two miles the effects are within the normal range of velocity, temperature and pressure resulting from our natural storms. Therefore, special consideration must be given only to the design of structures which because of civil or military importance must be made safe at explosion center distances of less than two miles. It appears to be practical to design steel structures to resist atomic explosions at distances of about one mile or more from the center with only minor damage. At this distance the structure would be subjected to a wind gust having an initial pressure of about 7 pounds per square inch gauge, arriving at a velocity of 200 miles per hour and lasting for a period of about 9/10 second, causing a wind force on flat surfaces of approximately 180 pounds per square foot, or six times the present design force of 30 pounds per square foot for a 100 mile per hour wind. The fact that this gust and the overpressure would last for a period of only about one second makes it a problem in dynamic rather than static design. The force of 180 pounds per square foot tends to accelerate the structure in a direction away from the blast, but the force is of such short duration that the heavy mass of the structure will offer effective resistance. Until more is known about the resistance of steel structures to large forces and high pressures of very short duration, designs for specific distances from atomic explosions cannot even be approximated. Steel structures more resistant to atomic explosions will probably be designed so that they will withstand a greater amount of deflection. This might be accomplished by providing diagonal tension members which will stretch during the initial gust and thereby absorb the acceleration imparted to the structure by the blast. These members can be replaceable after the blast. All enclosed structures will have to be provided with panels, in the roofs, floors or walls, which will blow either in or out, spaced close enough so that external and internal pressure will be sufficiently equalized to prevent damage during the rapidly changing pressure conditions. Paint and roofing must be fire resistant, and combustible materials must not be used where they will receive direct radiation from the blast. So much for atomic blasts.

For many years engineers have dreamed of tapered steel members, because in many cases the variation in intensity of the stresses is such that members which taper in depth or thickness would be most efficient. For example, the steel plates in a standpipe should economically increase in thickness with depth from the water surface. A cantilever beam should theoretically approach the curved outline of a parabola. Such members, however, are not suitable for production in rolling mills, as the very nature of this process requires that the sections be uniform in width and thickness. However, welding provides an economical method of modifying rolled sections so that they more effectively resist many types of loads. One method of making tapered beams is to split standard beams by shearing or burning diagonally through the web, reversing one half and rewelding the two halves together. Such tapered beams may be used in structures to attain savings in weight, improved appearance, and in many cases savings in cost. Tapered members are especially suitable for use in welded rigid frame and rigid knee structures, in jib crane booms and masts, in the cantilever roof supporting members of steel grandstand roofs, and in several types of bridges.

Figure 2 illustrates a jib crane constructed of tapered members in the manner I have described. This shows
ultimate strength to 65,000 pounds per square inch of actual area. A safe working stress would be over 30,000 pounds per square inch, and a 33 foot member made of this steel still has the capacity to stretch 30 inches before it breaks. I cannot visualize any possibility of ever requiring 30 inches of stretch in a 33 foot member in a steel structure, except possibly in special structures such as in the expandable tension bracing of towers subject to blasts or earthquakes.

Professor D. J. Lambert, of the State University of Iowa, has found that by prestretching reinforcing bars 10 per cent, the strength of a concrete beam or slab made from the bars may be increased up to 50 per cent and more.

Professor Ashton, also of the University of Iowa, demonstrated that initially stretched steel beams are up to 50 per cent stronger and have a higher factor of safety. His conclusions are that "Present standard requirements for ductility in the structural grades of steel are not the most desirable. Our structural grades could be used with greater efficiency if they were manufactured with the yield point closer to the ultimate strength."

Stretching is not the only means by which steel may be cold worked. In the example referred to, the steel was stretched after it had been rolled at the mill. Since rolling is essentially a stretching operation, the steel could just as well have been stretched at the mill by additional rolling. It is only necessary to use a proper finishing temperature to obtain any specified combinations of strength and ductility within the range of the particular steel used.

Cold work is not the only method of increasing the strength of steel. It may also be done by the addition of alloys or by the addition of carbon followed by a suitable heat treatment. By these means the strength of steel may be increased to many times that of the annealed steel we use in our industry. For example, the springs on your automobile probably have a yield strength of over 100,000 pounds per square inch and they serve their purpose much better than would soft annealed steel. In our daily lives we use much more stiffened, strengthened or hardened steel than the soft hot-rolled kind we specify in our industry. For example, you could not carve a steak, drill a hole, saw a board, or even make a satisfactory bobby pin out of hot-rolled soft steel. We do use soft steel effectively in stove pipe wire. The automobile and aviation industries specify most of their steels by carbon or alloy content and heat treatment to make them best suited to the job they have to do.

The present specifications of the Institute do not recognize any material for main members except ASTM A7, structural steel for bridges and buildings. Our specifications are inconsistent in this respect, for many structures are built under them in which high strength steels are used, for example, bridge cables which may have ultimate tensile strengths of 200,000 pounds or more, and high strength bolts which have come into recent use in both bridges and buildings. Steels for bridges and buildings, many of them suitable for welding, having higher strength than ASTM A7 have been available for many years and are occasionally used in structures where reduction of dead weight is important.

There has recently been developed a steel which I believe will be revolutionary in our industry. It is described in ASME paper 51-Pet-5 entitled "A New High Yield-Strength Alloy Steel for Welded Structures." This steel has an ultimate tensile strength of over 115,000 pounds per square inch, a yield point of over 100,000 pounds per square inch, and is suitable for a working stress of 50,000 pounds per square inch. It is easily weldable with low-hydrogen, high strength electrodes. It contains insignificant amounts of alloys, less than one per cent each of nickel, chromium, molybdenum and boron. The steel has high strength because it is a quenched and tempered steel which is alloyed in such a way that welding does not destroy the effect of the tempering. This is accomplished by slowing up the transition rate so that the duration of the weld heating and cooling cycle is shorter than the time required for transition. One of the most important things about this steel is that its price is lower per thousand pounds of strength than ordinary A7 steel. In addition to the saving in the cost of the steel, there is further saving from the reduced sizes of the welds. In both butt and fillet welds, the volume of weld metal is only about one-quarter as much as for A7 steel having equal strength, which results in appreciable savings in welding cost. This steel is, in my opinion, the most important development in structural steel in many years. Our specifications do not at present permit its use. It would be possible to immediately approve its use for tension members, but our present lack of knowledge of the effect of buckling and other elastic instabilities as intensified by the higher working stresses and the greater resulting deflections prompts us to defer recommending complete approval. We do not even know to what extent the compression flange of a beam depends for its strength on the l/r ratio. If it depends largely on column or local buckling strength, then the higher strength steel will not greatly increase the carrying strength of a beam. If the strength of the compression flange depends very little on the column strength, then the higher strength steel will considerably increase the carrying strength of a beam. Research work now under way will provide answers to some of these questions, but still more research must be completed before this and other high strength steel can be used with complete confidence.

In connection with built-up beams and girders, it would be possible to use high strength steel in the tension flange and ordinary steel in the compression flange. At first thought this seems like a good idea but after working
on it for some time I became convinced that this construction will never result in any appreciable saving of weight as compared with A7 steel in both flanges. I can recommend this problem as an interesting mathematical diversion.

In conclusion, I am of the opinion that the need for conserving our steel resources coupled with the new knowledge to come from the research work I have described will result in greater economy and in improved competitive position and at the same time retain and enhance the inherent strength and dependability of structural steel.

Chairman Jones: Thank you, Mr. Jackson, for a general survey of what is under way and the specific illustrations of big things that call for further research. I am sure that the things that Mr. Jackson has said are going to elicit some comments and questions. We agreed beforehand that we would present the three papers and have the comments and questions later. If you will please bear in mind questions that you want to ask Mr. Jackson, ask him later.

Mr. Jackson gave you an idea of the joint sponsorship of the research program being conducted at Lehigh University. That program has been running for several years and covers a rather broad field. The general question that it aims to cover is whether in continuous and therefore indeterminate structures there can be economic advantage taken of the actual collapse load of a structure in which not all of the parts are within the elastic limit as our ordinary design rules specify. Research of that nature is interesting to the Welding Society, because in welding we have apparently an excellent means of securing continuity. Therefore, the Welding Research Council has cooperated with this Institute and the other sponsors in finishing that research.

Mr. Beedle, who will be our next speaker and who is the Assistant Director of the Fritz Engineering Laboratory at Lehigh University where this work is being conducted, has been living with the work for some time. Last year the sponsors of the research were able to let him spend some months in England where Professor Baker has been out somewhat ahead of us in this field. In fact, Baker has arrived at conclusions that are actually being used in some instances in the construction of some structures. I want to present Mr. Lynn Beedle.

The Institute’s Research Program—Part II

Mr. Lynn S. Beedle

Gentlemen, I think that Mr. Jackson and Mr. Jones have given all the preliminaries to my talk that are necessary, so I will get right into it.

Introduction to Plastic Analysis

One of your published twelve advantages of Steel Construction is Toughness. Because of that toughness, or the ability to deform by yielding under overloads, it has been known for many years that continuous beams and frames will carry more load than that corresponding to the elastic limit. Testimony to this are the many steel buildings which, under bombings in Europe, have withstood tremendous overloads by deforming in what is called a "plastic" manner.

Who among you stand with the unique building authority in Germany who required during the 1920's that a two-span continuous beam, which had been designed continuous, actually be constructed as two simple beams? No, continuity is a source of reserve strength upon which the engineer relies.

Granted that steel frames are stronger than the elastic limit load, questions that are of immediate interest are:

(a) How much stronger are frames than the load which corresponds to the working load?
(b) Can we calculate this increase in strength?

For, if the increase in strength is considerable and if it can be predicted with reasonable certainty, then one is immediately led to the conclusion—

Can't some use be made of this reserve of strength in design?

Before attempting to answer any of these questions, perhaps it would be well to briefly review what it is that allows this increase in strength. It is the Plastic Hinge, a term introduced in Germany more than 25 years ago.

Figure 1. This curve of bending moment, $M$, plotted against curvature constitutes the basis for the "Simple Plastic Theory." At Point 1 the elastic limit has been reached; the stress-distribution is as shown. The curvature is designated as $\phi$, and is the relative rotation of two cross-sections a unit distance apart. At Point 2 the member is partially plastic. Point 3 is approached as a limit, termed the plastic hinge moment, $M_p$. As is evident from the stress-distribution for Point 3, it may be computed by multiplying the yield-point stress by the static moment of the entire cross-section, termed $Z$. Since free rotation occurs at this moment, the origin of the term Plastic Hinge is evident.

Since structural steel strain-hardens after a strain of about 15 times the elastic limit strain, the beam section will carry increased moment beyond the plastic hinge.
moment, and this will commence at a curvature of about 15 times the elastic limit curvature. Now, if we look at a continuous structure under load and keep in mind the \( M-\phi \) characteristic of structural steel it will be evident how the structure is able to carry a substantial increase in load.

Figure 2. This fixed-ended beam, loaded at the third-points, has a distribution of moment as shown by the dotted line when the elastic limit is reached. The end moment is twice that at the center. On the load-deflection curve we are at \( "P_y" \). As the load is increased, the plastic hinges form, and with further application of load the beam behaves as if it were simply-supported, except that constant and moments \( M_y \) are acting at the ends. This continues until the initial yield moment is reached at the center panel (under uniform moment). The beam then goes into the final plastic stage and the ultimate load, \( P_y \), corresponds to the moment diagram at collapse. The reserve of strength is clear — in this case 50%.

An additional concept must be introduced, rotation capacity. It may not be enough that the beam or connection or column be able to sustain the plastic hinge moment. Especially in the case of the first hinge to form, is it essential that the section be able to rotate through a considerable unit angle change after reaching the plastic moment in order to develop plastic hinges elsewhere.

Fig. 3. Thus in this \( M-\phi \) curve, if line A represents the simple plastic theory and \( \phi_A \) the required rotation capacity to develop all hinges, structural components acting like B and D would not be doing their job. The one is not strong enough and the second collapses before the necessary rotation has been developed. Member C would meet all of the requirements.

With a big IF, then, the first questions of interest are answered. We can calculate the increased strength of frames — if the component parts have the proper moment-rotation characteristics.

Thus, it appears that steel can be saved in the design of continuous structures. (The word "continuous" is emphasized because plastic analysis has no place in the consideration of determinate structures.) It is not at all difficult to show savings of 20% or more in weight of main material by utilizing the reserve of strength. I am intentionally careful not to say PLASTIC DESIGN and thereby spell out a particular design method. At this time I wish to emphasize that it is not our purpose at Lehigh to advocate or promote any particular design method; rather it is to study the plastic behavior of structural beams, columns, connections, and frames, to observe their reserve of strength, and to explore limitations that may be involved in the utilization of the plastic range in design.

There are numerous ways advocated for making use of the reserve strength. We have treated one such which will be used in an illustrative example. The important thing is that the design method be simple, that it treat the problem in a rational manner, and that any limitation relating to the physical behavior of the material be fully recognized.

Figure 4. Let’s take a concentrated load off-center on a thirty-foot span. Assume a working load of 21 kips. If the ends are simply-supported an 18WF50 shape is required. The elastic limit is the implied “Full Load,” or working load multiplied by the factor of safety.

If the ends are “fixed” against rotation, according to conventional elastic design procedure a 16WF36 shape is indicated. The load at initial yielding is the same as in case I, but obviously the reserve in strength is considerably greater.

Finally, if we use one of the available plastic design procedures, the weight can be reduced an additional 20% (14WF30). The example shown makes use of one of the most conservative plastic methods yet suggested. The things to note about the results are:

a) The full load (working load times factor of safety), for Case III has the same rational basis as Case I — the load at which deflections commence to increase at an uncontrolled rate.

b) At working loads the structure is still in the so-called "elastic range."

c) The deflections at working loads for the plastic design are less than those of the simply-supported beam and only slightly greater than the simple beam at full load. Also note that it is well within the specification limit.

Lest you consider that the computation of these deflections is a tedious proposition, examine the load-deflection curve for the same example.

Figure 5. The curve has three approximately straight portions joined by short curves. In the elastic range, the deflection under load can be determined from the handbook. Starting from point A, the segment AB represents the load-deflection curve of the beam in sketch (b) loaded within the elastic range. Likewise the load-deflection curve of the beam in sketch (c) will be similar to the portion BC. Both of these relationships may of course be determined from the handbook.

Well, we are not here to consider the merits of different plastic design procedures. But this serves to illustrate possible savings. Rather, we are concerned, first, with analysis and answering the big "IF": do structural members have the proper moment-rotation characteristics to make it possible for us to calculate dependable reserve strength in continuous beams and frames?

**The Behavior of Structural Components and Frames**

One of the unique features of the investigation at Lehigh is that it emphasizes the testing of full-size rolled structural steel members.
FIGURE 6

FIGURE 7

FIGURE 8

FIGURE 9

THEORETICAL AND EXPERIMENTAL MOMENT - $\phi$ CURVES FOR 8WF67 BEAMS

Theoretical curve for annealed beam.
Theoretical curve for as delivered beam.
Experimental curves.
Calculated initial yield

Beam 2  Beam 3  Beam 4  Beam 5

Curvature in Radians per inch (Strain Gage Data)

FIGURE 10

Calculated initial yield

Moment in Inch-Kips

Theoretical Curve
Experimental Curve of Beam - 3
Experimental Curve of Beam - 4
Experimental Curve of Beam - 5

φ in Radians per Inch

FIGURE 11

BEEDLE
Curvature in Radians per inch (Strain Gage Data)

**FIGURE 12**

**FIGURE 13**
Figure 14

Figure 15

BEEDLE
FIGURE 16

BEEDLE
Figure 6. This shows the test of a beam, continuous over two supports, loads being applied at the ends and the center span third-points. The shape is 14WF30 and the member is of course still elastic. Figure 7. This shows a single-span portal frame under vertical load at the three-eighths points. The shape used is 8B13.

Figure 8. It is certainly evident here that steel is tough. This portal frame of 8WF40 shape has a center deflection almost 10% of the span length of fourteen feet — and the frame is still carrying increased loads.

Standard fabrication operations are used and members are invariably tested in the as-delivered, as-welded condition, so that residual stresses due both to welding and to cooling after rolling are present.

Do Beams, Columns, and Connections Develop Plastic Hinges?

Figure 9. These are M-ψ curves (moment plotted vertically and curvature horizontally) for simply-supported beams. The theoretical curves, based on coupon test results, are the same as those shown on our first slide. Excellent experimental agreement was obtained for the annealed specimen. Although the as-delivered beam is stronger than the annealed one, it does not quite develop the plastic hinge moment. Similar results have been obtained in 8WF31 and 8WF40 beam tests.

Figure 10. Here are M-ψ curves for four different beams. This dashed line is our same theoretical curve and these solid ones are experimental values. As-delivered specimens consistently show a reduction of about 10% in moment capacity in the early part of the plastic range.

Figure 11. However, when straining is continued and the yielding penetrates closer to the neutral axis, the moment capacity approaches the predicted value. Very recent data shows that this section also develops its predicted strength in the strain-hardening region.

Thus, if the straining is carried far enough then it is possible for the section to develop the plastic hinge moment.

Figure 12. Naturally some difficulties are encountered. Here is an M-ψ curve for a 14WF30 shape, a cross-section different from that shown in the previous slide. With the relatively deeper web and thinner flange, there is less restraint against local buckling. In this test which simulated a cantilever beam, the plastic hinge moment was not reached. Since this shape is the lightest in the 14-in., series and since, as the slide shows, a fairly good combination of plastic strength and rotation capacity were realized, it appears that an adequate number of shapes will be available when the time comes to utilize the plastic range in design. The next slide is a view at the "connection."

Figure 13. This shows the local buckling causing collapse of the cantilever. Local instability in the plastic range does constitute a possible limitation that is being carefully studied, at the present time, with emphasis on specifying those geometric proportions that will provide adequate strength and rotation capacity.

Figure 14. We will now examine the behavior of columns. In this figure, moment is plotted against end angle change for a column loaded in the same manner as one in a single span portal frame. The ratio of axial to bending stress is of the order of that found in most portal frames. The maximum strength of the column not only exceeded the predicted yield moment but exceeded the plastic hinge moment that would have been developed had there been no axial load.

The significance of this: The influence of axial load in portal frame columns may be neglected in many cases when computing the reserve moment capacity.

Figure 15. This shows how the column finally failed — local buckling of the most-compressed flange at the end — where the moment was a maximum. The inset shows the loading on the column and the position of the view.

Do Welded Continuous Connections Have Satisfactory Moment-Rotation Characteristics?

Figure 16. A series of connections for portal frames were tested in the manner shown in the inset. As is seen, the series includes a representative sampling of the major connection types, straight knees, and knees with tapered haunches and with curved haunches. Of course, these sketches are only diagrammatic of the proportions.

This composite M-ψ curve shows that straight connections (lower curves) can be proportioned simply and with economy to develop considerably in excess of the required moment strength and rotation capacity. The heavy solid line is the curve determined for the beam of similar shape.

These (middle curves) are for the tapered connections, while the strongest of the series were the curved knees (upper curves), proportioned according to the procedures recommended by the AISC.

Without exception all connections eventually collapsed by local and lateral instability. In the larger connections the collapse is more sudden; but this need not be a difficulty since frames using such haunches are usually so proportioned that no rotation capacity is required to develop the maximum strength. The critical sections of the frame would reach the yield point simultaneously. On the other hand, if, for some reason, rotation capacity were desired at a haunch, the knee proper could be made slightly stronger than required by elastic design so that yielding would first occur in the rolled section adjacent to the haunch — where good plastic properties could be assured. The next slide is connection L.

Figure 17. This is one of the straight connections (connection L shown in Figure 16) at the end of the
Prior to this, it had developed more than adequate moment and rotation value. Here again, final failure was due to local instability. It is an 8B13 shape, the corner being supplied with both vertical and diagonal stiffeners.

Thus we see that beams and straight connections under most circumstances will develop plastic hinges; columns will develop them in many cases.

Having reviewed the behavior of these components, let us look at the behavior of some continuous beams which depend on the formation of plastic hinges for their reserve of strength.

Figure 18. These are load-deflection curves of three continuous beams with three different welded details at the supports. The theoretical curves are shown by the dotted lines. We see that at as low as 20% of the predicted yield load, initial non-linear behavior is observed. This is due to stress concentrations and residual stresses, and the only result is to increase the deflection. Eventually all three beams developed their predicted ultimate strength. Another thing to observe is that there is no real significance to the theoretical initial yield point—except as it may provide an inherent deflection limitation. It indicates that deformation (deflection) rather than stress is the important criterion in structural design.

Figure 19 shows the "plastic hinge" at the support point of one of these continuous beam tests. And Figure 20 shows the "plastic hinge" that occurred at the center of the same beam.

Earlier the influence of cross-sectional shape was mentioned. Two complete portal frames were tested, one with the 8WF40 shape and one with 8B13 shape (similar geometrically to 14WF30).

Figure 21 shows how the frames were tested with hydraulic jacks applying the load at the three-eighths points.

Figure 22 shows the center portions of the two frames which are under pure bending moment. The more compact 8WF40 (at the left) has obviously deformed more under load than the light 8B13 shape (at the right) which has deformed locally before reaching the plastic moment or developing the full yield pattern in the web. The resulting difference in behavior is shown in the next chart.

Figure 23. This shows the load-deflection curves for the two frames. Frame 1 (8WF40) carries load considerably in excess of the predicted collapse load. This is due primarily to strain-hardening. Since the effect is neglected when computing the reserve of strength, it is, for many shapes, a further additional reserve upon which even the plastic methods of analysis do not rely. This is the record from the severely deformed frame we saw in one of the first photographs. It is evident here that a great deal of energy has been absorbed.

Frame 2 (8B13) almost develops the predicted collapse strength in spite of the local buckling in the plastic range. This is gratifying since it is difficult to find a shape in the handbook with poorer "local buckling" characteristics. The chief difference is in energy absorption—considerably less in this case.

Thus we are led to conclude that further consideration needs to be given to making use of the reserve of strength in indeterminate structures. The usefulness of plasticity in structural design is limited by the same factors which cause a modification in present-day elastic design—factors such as brittle fracture and fatigue, when these factors are present. When these factors are absent, structures should be designed on the basis of limiting the deflection rather than upon limiting the stress. Evidence indicates that in numerous cases this may be done simply, without exceeding permissible deflections or exceeding yield-point stress at working loads.

Although at least one structure has been designed in England according to the application of plastic theory, it is considered that its use in this country ought to await the further study of some limitations, local buckling being one of the most important.

Chairman Jones: Thank you, Mr. Beedle. I am sure that Mr. Beedle will also be on the receiving end of some questions later.

The University of Illinois has for some many years been in the forefront of structural research. They have a magnificent laboratory and staff there. It was there—it seems like yesterday but it is a dozen years ago—that they did the first work on fatigue strength in connections, first riveted and then welded. Work has also been going on at Illinois under the auspices of the Army and Navy and civil bodies, and a considerable amount of work has been done by the Research Council on Riveted and Bolted Structural Joints.

Dr. Newmark is the Director of that experiment station. He is Professor of Structural Engineering at that experiment station and he will give us a survey of the status of research on riveted and bolted structural joints. Dr. Newmark.

The Institute’s Research Program—Part III

Dr. Nathan M. Newmark

The program on riveted and bolted structural joints, which is being in part supported by the AISC, under the direction and coordination of the Research Council on Riveted and Bolted Structural Joints, has eight active
project committees engaged in different aspects of the program of the Council.

I am going to try this afternoon to summarize just a few items and to indicate the present status of some of the programs which are of major interest to fabricators and designers in structural steel.

The programs which are under investigation by the Council concern the effect of rivet bearing on the static and fatigue strength of plates of riveted joints, and Mr. Jones is Chairman of that particular project committee.

There is also a project on the effect of rivet pattern on the static strength or on the effect of net sections, if you wish, of riveted joints.

There is a project on the strength of rivets in combined shear and tension of which Mr. Higgins is chairman.

There is a program on the strength of bolted structural joints, and you have had a discussion of this program two years ago by Mr. Stewart, who is the Chairman of this particular committee and also of the Research Council at the present time.

The first three programs are being conducted at the University of Illinois. The fourth one, on bolted joints, is being conducted at both Purdue University and the University of Washington.

There are three programs going on at Northwestern University, on the effect of grip upon the fatigue strength of riveted and bolted joints, the fatigue strength of high-strength steel riveted joints, and the effect of rivet pattern upon the fatigue strength of structural joints.

Then there has just been established a new group to work on the effect of cumulative fatigue damage in structural joints. This program is still in the planning stages.

One can see from just the general description of the work of the Council that the program is directed toward immediate practical application as well as to long-range fundamental studies. Some of the programs will lead to proposals for revisions in current specifications within a very short time. Others are concerned with studies which may have an effect on practice within several years and only after a good deal of further work. Those of us in close association with the Council's work feel that we have achieved an excellent background between the two principal objectives, both short-range and long-range fundamental studies. We have been able to do so because of the efforts of men like Mr. Jones and Mr. Higgins who have been closely associated with the work of the Council since its beginning.

I would like to summarize some of the work of just three of the programs. The first series of slides which I am going to show concerns the results of the study of the strength of rivets in combined shear and tension. Current specifications do not in general make provisions for rivets subjected to such combined stresses.

The Research Council considered this matter to be an important one and set up a project committee to study it. The first development was in the direction of determining a satisfactory test specimen. Then a pilot series of tests was undertaken to make a quick survey of important variables. A more elaborate program was planned finally to cover all variables such as the rivet grip, rivet diameter, method of driving, and method of manufacture of rivets, in addition to the strength under various ratios of shear to tension.

Figure 1 shows the test specimen which is a rivet driven into a jig consisting of two parts with shoulders against which pull could be applied or on the sides of which shearing forces could be applied. The specimens were individual rivet specimens.

Figure 2 indicates a cut-away view of the test jig in the apparatus showing the method of gripping test specimens for conducting the test.

Figure 3 shows the apparatus in position for a test in direct tension on the specimens. If it is rotated 90 degrees the pull comes transverse to the rivets and shear load is put on the specimens. By pulling at the various other pairs of holes different ratios between shear and tension can be achieved.

Figure 4 shows typical fractures of rivets in pure tension, pure shear, and intermediate ratios of tension to shear. One can note that the reduction in area is greater with a larger amount of tension, with practically no reduction in area for pure shear, and that the type of failure and amount of distortion, vary with the ratio of tension to shear.

Figure 5 gives some of the load-deformation characteristics. The top curve, No. 4, is for a specimen with pure tension, No. 1 with pure shear, and No. 2 and No. 3 with different intermediate ratios. The energy absorption in the tension specimen is somewhat greater than for these tested under shear load.

Figure 6 is a summary of the results of the preliminary tests plotted in two different ways. This method of plotting I have attributed to Mr. Higgins although he says it was developed before him. The horizontal plot shows the ratio of the shear component to the rivet tensile strength or to the rivet shear strength, one curve representing one and the other the other. The vertical elements represent the tension components in relationship to shear strength or tensile strength.

The inside ellipse shows the tensile component equal to tensile strength, and the shear component at failure, at about 70 per cent of the tensile strength. The intermediate values obtained, fall roughly under this curve.

The outer curve is plotted on a slightly different basis. The shear component here is plotted at 100 per cent when there is no tension, and the tensile component producing failure when there is no shear is at about 140 per cent of the shear strength for this particular series. The intermediate values obtained are as indicated.
Figure 7 shows the results of a large number of tests, some 200 or more in all. In the final program, where different rivet lengths, rivet diameters, methods of manufacture and methods of driving were taken into account, the points fit very close to the ellipse. The enlarged sections show the scatter in points about the ellipse. The variation is 3 to 5 per cent for the greatest scatter. The indications are that the ellipse is a very good representation of the strength under combined stresses.

The ratio of the tensile component in pure tension to the rivet shear strength is about 1.33, identical with the value given in the A.I.S.C. specification. The indications are rather interesting.

Figure 8, the last slide of this series, is a tabulation of the strength based on the hole area. With a shear-tension ratio of 1 to 0, that is pure shear, the ultimate stress is 44.9 kips per square inch. In pure tension the ultimate stress is 59.5 ksi, and intermediate values for intermediate cases are given. The ratio of the ultimate stress in tension to that in shear is 1.324, which I submit is fairly close to specification allowance.

One way of looking at this result might be of interest. If we have a group of rivets subjected to some tension and to some shear, the problem is to determine how many rivets are required to carry the combined load. One can approach the problem roughly in this way, which is, of course, only an approximation since we do not usually have such combinations where all of the group of rivets are subjected to the same conditions. But if, for example, we require \( N_s \) rivets for shear, based on specification allowance of 15,000 pounds per square inch, and \( N_t \) rivets for tension, based on allowance of 20,000 pounds per square inch, the number we would require for a combination of these stresses would be:

\[
\sqrt{(N_s)^2 + (N_t)^2}
\]

For example if we had to carry 80,000 pounds, let us say, in tension, and 45,000 in shear, we would need 4 square inches for tension, 3 square inches for shear, or a total of 5 square inches of rivets for the combined load.

With these results, the Council has not completed its work on this program, of course. The next step is to make the results available to practicing engineers. On the basis of these tests and results, and taking into account experience of practicing engineers the Council expects to make recommendations for design specification provisions that will cover the combined effect of shear and tension on rivets.

The first complete report of this program was issued two months ago, and a short summary of the results will be published, in the near future, by Professor Munse who is in charge of the work at Illinois and Mr. Higgins. I would like to call your attention to the fact that here is an instance where the results of research are almost immediately suitable for use in practice. It illustrates an important aspect of the organization of this Research Council. The results are made available to that segment of the profession, at least, which is represented on the Council, and the research workers have the benefit of advice and suggestions from these important representatives of the engineering profession. When a report of the work of the Council is published, it has the backing of a good part of the people who will use the results, since these people are represented on the Research Council.

In general, the lag between the results of the research and the practical application is made as short as possible because of the way in which the Council is organized.

The next series of slides show the results of another topic, that is the effect of rivet pattern on the net section of riveted joints. This series of tests, which I am going to show you, also indicates a development of a means by which many of the variables that caused discrepancies in previous test results were eliminated. This was done by the use of small-size specimens which could, in general, be made from the same plates, so we could eliminate the effects of variation of materials. We had to justify this, and it is indicated in one of the slides which will follow later.

Figure 9 merely is an indication of three rivet patterns among some 50 or more which were tested, and is given only to show a range in the results. The efficiencies of these three joints by tests are shown in the lower line, and by the current design rule which is based on Cochran's formula in the upper line. You will note that for the specimen on the left the test result is about 8 per cent greater than the specification rule. In the middle one it is exactly equal, and on the right hand side some 6 per cent less.

The results of three tests, which were made by Mr. F. W. Schutz, indicate a theoretical value of 82.6 per cent, compared with 81.5 for the left hand specimen, and 87.0 for the other two, or within about 1 per cent of the test values. Of course, he had the advantage of having these tests on which to base his theory so we can't claim a good deal for this result, however, we can show later that the results of a great many other tests also fall within this range, and there is some reason for believing that Mr. Schutz has a workable and reasonable answer.

The specimens were fabricated in such a way that they could be put into the testing machine quickly, with the ends made in such a way that they could be put into "T"-shaped slots so that a good many specimens could be tested. Over 200 specimens were tested in a period of 4 to 6 months.

Figure 10 indicates the type of failure in 3 different series of specimens all to identically the same scale. The smallest one is 3 inches, the intermediate one in the
middle is 6 and the largest one is 9 inches wide. The full-size fasteners in the largest specimen were \(\frac{11}{2}\)th inch rivets and the fasteners in the two smaller specimens were bolts of scale size tightened to about the same degree of tightness as the rivets.

All three specimens are pictured laid one over the other, indicating the general shape of fractured surface which is the same. The net section was almost the ratio of strength to coupon strength, and was almost identically the same on the 3 specimens.

Several other simple tests of this sort were made to justify the use of a small-size specimen for the major part of the series.

Figure 11 shows the results of Mr. Schutz's test and of all of the tests which Mr. Schutz was able to collect, which pertain to riveted joints in which the rivets are driven in drilled holes, and in which the holes have a rectangular pattern but with different ratios of pitch to gage, different spaces, edge distances, and so forth—other variables. The curves in general plot fairly smoothly in a plot of gage distance against diameter of rivet, and as this ratio changes from very small values on to fairly large ones, the efficiency in per cent goes upward from something of the order of 57 per cent up to something of the order of 87 per cent. At the ratio of about four, or four and one-half, one achieves the maximum efficiency possible. At smaller ratios, one gets a lower efficiency.

One can define the gage as being, either the shortest spacing between adjacent rivets in the outer line, or twice the distance from a rivet to the edge of the plate in that outer line. With this definition, the points all fall in a band, plus or minus 5 to 7 per cent from the average curve.

The tests which Mr. Schutz made under fairly carefully controlled conditions show a scatter of less than 3 per cent from the average curve.

Figure 12 indicates all of the other data which could not be classified into drilled holes, or for which we were not sure of the type of holes. These were in some cases punched holes and in other cases sub-punched and reamed. In general, where holes are punched the points fall well below the average values for drilled holes. Some of the points which were unidentified probably were for drilled holes and do collect along the previous curve. The curve shown here is more or less a minimum curve for all of the other tests, but the same general relationship is obtained. There is an indication of a decrease in efficiency of the order of 10 per cent for punched holes compared with drilled holes. Incidentally, the test results which were plotted here include tests made at the Water-town Arsenal, 1882 to 1900.

The University of California tests are included as reported in Transactions of ASCE 1940. Also included are all of the tests at the University of Illinois, from the time when the first series was made in 1932 under Professor Wilson's direction, some tests by Otto Graf in Germany, and the tests at Northwestern University on projects 5 and 7. All of these fall on the same curves.

The next short series of slides summarizes the results of the program of tests to determine the effect of high bearing stress. The objective of these tests was to determine whether bearing ratios of the order of two or more could be used in design without serious reduction in strength. The indications of the tests, as you will probably note from the slides, are that for static loading there is no decrease in strength and possibly even a slight increase up to varying ratios of slightly greater than two. However, further studies are required to investigate the possible damaging action of high bearing ratios in fatigue. Nevertheless, the results of studies to date indicate that the present specification which allowing a bearing stress of twice the tensile strength is adequate and reasonable and perfectly safe.

Figure 13 shows the typical failure for three specimens (50-8A, 50-8B, and 50-8C) of a bearing ratio of only 1.35. They are identical specimens but they failed in two ways. In 50-8B, one outer plate failed through net section; the other side sheared through the rivets. Specimen 50-8C failed by shear in the rivets. The last specimen on the picture (50-7A) is for a bearing ratio of 2.75 where there is evidence of some tearing because of the high bearing stress on the outer row of rivets, although the rivets failed in shear.

The ratio of shear to tension was about .75. The indication of this test and others that we have conducted is that this is just about a balanced ratio. There is a slightly greater tendency for plates to fail in tension than in shear of fasteners with this ratio of shear to tensile stress, but some specimens do fail by shear of fasteners at that ratio.

Figure 14 indicates the variation of net efficiency, computed as the ratio of ultimate stress of the net section to the coupon stress, in terms of bearing ratio. You will note that for low values of the bearing ratio, the efficiency is greater than 100 per cent which arises because of the fact that the constriction due to the bearing ratio does not permit as much reduction in cross-section as occurs in a tensile stress coupon.

Figure 15 indicates the gross efficiency, or efficiency in terms of the gross section, which does show the increase, strange as it may seem, up to a bearing ratio of two or slightly higher, and then possibly a slight decrease. The theoretical value shows an increase throughout since it does not take into account the effect of the higher bearing stress on the tendency to develop a fracture in the plate.

The next group of slides summarizes just a few of the results of static and fatigue tests on bolted joints. I don't have time to give a complete survey of the work on it at this time, but a symposium on this topic will be held at
the ASCE Centennial Meeting in September and a number of papers will be presented.

Figure 16 shows the details of specimens for static tests of bolted joints. It shows single lap joints with two bolts, and three bolts, and a double lap or butt joint with two bolts.

Figure 17 indicates results of the tests of specimens. The joints were designed so as to have shear-tension ratios corresponding to 0.75, 1.0 and 1.25. At the lower values of shear-tension ratio, the failures were in the plate at tensile stresses of around 64 to 67 Kips per square inch. In one instance, at a shear-tension ratio of 1 in the butt joints, one of the specimens failed in shear and one in tension and at about the same tensile strength in the plate, indicating that this may have been a balanced design. There was probably something else wrong although we are not sure what, because in no other case did we get a shear or fastener failure at such a low ratio.

In the three groups having a ratio of 1.25 to 1, the stress in the plate at failure in tension was about the same as the stress in the plate for failure in shear. Some of the failures were of each type. The indication is that balanced design is achieved at about this point because of the higher strength of the fastener. Balanced design in a riveted joint at a shear-tension ratio of 0.75, and balanced design for shear in the fasteners is reached at a shear-tension value of 1.25, which indicates that we can replace rivets by a smaller number of bolts if we are interested only in ultimate strength. If we must prevent slip, we may still be able to replace rivets by a smaller number of bolts, but we have not yet enough data to decide how many fewer bolts we can use.

Figure 18 is another indication of the same results, and shows comparisons of similar specimens made with riveted joints where all of the failures were plate failures at 0.75 and all the failures were fastener failures at 1 and 1.25 for riveted joints. In bolted joints, there was one failure of fasteners at ratio of 1 and about the same number of failures in plate and fasteners at the higher ratio.

Figure 19 is a summary of values for co-efficient of friction between the plates of a joint connected with high strength bolts. This is a summary of tests made at the University of Washington by Dr. Hechtman. There are a great many specimens summarized here, different patterns, different numbers of bolts. The indication is, however, rather interesting. The minimum co-efficient of friction in terms of shear, effective shear strength, in its ratio to the fastener tension, ranges from about 23 per cent minimum up to a little over 50 per cent, with an average of slightly over 30 per cent. This indicates that we would not expect slip to occur until values well over designed stresses are reached in a joint in which the rivets are replaced by an equal number of bolts, of the same effective diameter.

We have some results at Illinois and at Northwestern which give substantially the same results, somewhat higher for the maximum values, but the same for minimum and slightly lower for average, running about 25 per cent co-efficient of friction. However, this does not change the conclusion particularly.

Figure 20 concerns a series of tests of fatigue of bolted lap joints with four bolts in a rectangular pattern. There are a number of different variabilities involved here. Specimens A-1, A-2 and A-4 had as variability the tension in bolts, which ranged in steps from 50 per cent of elastic proof load upward to 138 per cent of elastic proof load for A-4. This produced about as much elongation in the bolt as we could count on without snapping the bolt in two. When we tried to produce larger values, the bolts failed. We made these tests with large values to find out whether there would be any tendency for the bolt to fail because of repeated loading, but in no case did we get a bolt failure.

Figure 21 gives the results of the fatigue tests. There isn't much really to see in this because there were only two cases where fatigue failures occurred, A-1 and B-3-R. In all the others, at complete reversal of stress on the net section, of plus and minus 18,000 pounds per square inch, slip did not occur and fatigue failure did not occur. A-1 which had only 50 per cent of the elastic proof load did not have enough resistance to slip, because the fastener tension was too low, and failure occurred at 1.2 millicycles of stress. Specimen B-3-R, the riveted joints, carried only about one-half a millicycle of stress. All of these joints might be considered to be joints in which rivets are replaced with an equal number of bolts. The fatigue stress is increased by a tremendous margin. Even for an insufficient fastener tension, the fatigue stress is still increased over that for a riveted joint. I would like to call attention particularly to A-4 where the high fastener tension does not affect fatigue strength. No peculiarities in behavior of this specimen were noted.

The last series of slides, which I am going to show, indicate the results of a program to determine if longer fatigue lives are possible with some of the low-alloy steels. The use of A-7 steel has some advantages, it has been felt previously, because of the great fatigue resistance even though the static strength is not high compared with alloy steels. However, the Council determined that this point should be re-examined since the previous tests were few in number and only two steels, nickel and silicon steel were compared in the previous test.

So, at Northwestern University a great number of different alloy steels, something of the order of 15 different types, having the different stress-strain diagrams shown in Figure 22, were tested in comparison with A-7 steels to determine the relative fatigue strength under conditions representative of those that might be found in a bolted or riveted joint. The test conditions may have
been too favorable. The work is still going on.

The test specimens consisted of a plate with a machined or drilled hole in it. It may be that there would be a different result if an actual joint were tested. As a result of this first series of tests, several particular alloy steels were chosen with which to make tests of actual joints. These are to be made soon and the results will be available, we hope, within the next year or so.

Figure 23 indicates the nature of the preliminary results for carbon and rim steels. These are all curves of stress against number of cycles, something of the order of 24 to 26 thousand pounds per square inch at one millicycle of stress, in zero to tension.

Figure 24 shows the results for silicon steel and can serve as a basis for carbon. One of the average curves for the A-7 steels is shown here, and there is an indication of an increased endurance of the order of 32,000 psi at one millicycle.

Figures 25 and 26 are for some of the low-alloy steels. Steel G has quite a material advantage over the other steels. Its endurance at one millicycle is the order of 34 or 35 thousand psi. There is some indication of promising results here which will be further investigated.

The research programs that I have described cover only part of the work of the Council. However, I think, enough of the program has been described to indicate the general nature of this work and the importance of the results achieved to date. Although the Council has been in operation for only a few years, important practical results are now available and should find their way into publication almost immediately. Other practical applications will develop out of current and proposed future research. The results of the Council's work, we hope, will be to attain greater efficiency or greater economy in the use of structural steel, benefiting the fabricator, the designing engineer, and the general public.

CHAIRMAN JONES: Thank you, Dr. Newmark. I think you did a splendid job.

Now, if the speakers will reassemble, and get prepared for questions the audience will fire, I will open with the project on the effect of bearing pressure on riveted joints. Dr. Newmark said that these tests had indicated that a bearing to tension ratio not over two would make no reduction in the strength of the joints. He slipped a little in implying that that was in all present specifications. The present bridge specification has a ratio of one and one-half. The tension permitted is 18,000, the bearing 27,000. This research has shown, I feel, that that bearing stress could be 36,000 and there would be no possibility of the joint being weakened by that increase. Now, it really shows a little more than that. It can be carried a little farther than that.

Dr. Newmark mentioned that in the other projects which Mr. Schutz has been carrying out, things are being discovered as to the pitch-gage relationship which makes for the greatest efficiency in the development of a tension member. If you tie that into the program on bearing, you will find that when those relationships of pitch and gage, and of rivet shear to plate tension that create economy are observed, then the bearing pressures automatically never will be high enough to be taken into account or specified at all. There is just one exception, and it is not an important exception. If you have only a single line of rivets transverse to your joints and they are in double shear, then this ratio of two times tension, or 36,000 in bridge specifications, does appear to be a stopper. That is the only instance in which it is a stopper and that is an unusual situation, a single transverse line of rivets subjected to a computed double shear. So, the general results of the research on bearing pressure appear to indicate that bearing can be almost forgotten.

Now, the next question is: Is it worth finding that out? Probably in bridge work it is not of great importance because the material there is usually thick enough that rivets are not added, nor is the material thickened up because of the specified 27,000 bearing. In building construction that is not true, because you get into the use of thin material. And when you get the relationship between a three-quarter inch rivet and a plate, you have a relationship in which the bearing restriction does come into account.

We had quite a bit of experience with one job in which a connection of that general relationship between the size of the rivet and the thickness of material, was multiplied into thousands of connections over the entire project. In a situation like that, it should be of great advantage if we can establish the fact that you can forget the bearing—that if you take care of the relationship between the rivet shear and plate tension, you have done the job. Not that I expect to see the bearing requirement thrown out of specifications, but I do believe we will justifiably raise the value to a point where it will be automatically forgotten even by designers, because it will not hamper them at all.

The meeting is open for a discussion of all three of the papers. As usual, I will ask that you give your name to the stenographer and mention to which of the speakers you are addressing the question.

MR. ENEY (Lehigh University): May I see Figure 13 of Mr. Beedle's presentation? I have in mind the slide that showed the local buckling in the compression flange of the 8B13.

The question is, assuming that there would be some tendency to adopt this "plastic hinge" type of design, and recognizing that it is not likely that rolled sections can be repositioned on those types of frames, where our loading is not going to move, would we be able to raise the ultimate strength and the hinge value if it were
possible to thicken up the edge of that compression flange in one of several ways, such as welding, a short edge stiffener, tapered out to prevent stress concentration? Or would that mean that local buckling would occur at a lower bending moment out along the beam and nothing would be gained for the extra expense?

Mr. Beedle: I think that is probably the case. If you attempted to stiffen the compression flange locally at a point where it would tend to buckle, it would merely buckle at some point a little farther from the connection. I expect the expense would be more than would be gained in improved rotation characteristics.

Thank you for getting me on my feet because I want to acknowledge here the great amount of help that we have received on our project at Lehigh from members of the Lehigh project subcommittee. Among those in the room who were on that committee are: Dr. C. E. Webb, Mr. Jones, Dr. Newmark, Mr. Carl Kreidler, Mr. La-Motte Grover, Mr. Weiskopf, and Dr. Bruce Johnston, now at University of Michigan but formerly Director of Fritz Laboratory at Lehigh and also of this project. And particularly, I want to acknowledge the help of the Chairman of that committee, Mr. Higgins.

Mr. Beedle: This Figure 24, (Refer to Discussion—Part II), shows about half of the sections that were tested and I think has the detail better than the photograph. One of the stiffeners wasn’t a stiffener. The connection is between two rolled flange, not a stiffener. Then the problem that was studied was to see what influence those fabrication details would have. The second question is: Why did you insert horizontal and vertical stiffeners in view of the fact that you had a diagonal?

Mr. Beedle: This Figure 24, (Refer to Discussion—Part II), shows about half of the sections that were tested and I think has the detail better than the photograph. One of the stiffeners wasn’t a stiffener. The connection is between two rolled shapes, one of which is butt welded to the other so that one of the parts is a flange, not a stiffener. Then the problem that was studied was to see how much effect different depths of stiffeners would make on behavior, so a half-depth stiffener was used, and a full depth was used, and another connection was tested with no stiffener at all to extend the inner column flange. Connection L is the one we saw in the slide.

The purpose of the diagonal stiffener was to counteract some deficiency in web thickness. Previously we had tested a connection which had commenced to yield, as connection P. You see, this has no diagonal stiffening at all, and the results of that, are shown in the slide, that we will come to in a minute. After we had tested connection P and found that it yielded a little more than desirable, then the four different alternates were tested to see what influence those fabrication details would have.

Mr. Foehl: What results did you find? How did that last one, with just a single diagonal stiffener, compare with the one where you used the vertical and horizontal stiffeners? What I am getting at is that, in my own mind, if you put in a good diagonal you don’t need anything else.

Mr. Beedle: That is true in the elastic range, and according to conventional elastic design it would behave perfectly well. In order to make sure it would develop good characteristics in the plastic range, it appears in the tests that the principal advantage of vertical and horizontal stiffeners is that they supply stiffening to the web. So, connection A, the one with the diagonal stiffener, has a tendency to have poor rotation capacity. That is, after it reaches the maximum it collapses more rapidly. That is only a point, to consider if you are starting to consider the use of plastic ranges in design. Connection A has one possible drawback from the economic point of view in that it might require special facilities for making that 45 degree cut if a large number are to be fabricated. Normally, one would cut a beam straight across.

Mr. Griffiths (A.I.S.C.): You indicated that you had another slide showing connection P. I wonder if you might show that.

Mr. Beedle: In Figure 25 (Refer to Discussion—Part II), the solid line is the experimental M-phi curve. This connection joined two different shapes of cross-section, an 8WF31 and a 14WF30, with no stiffening diagonal. The dotted line is the M-phi relationship for one and the dash-dot line is for the other. At a bending moment of half the capacity of the lighter member, the connection started to deform due to shear yield in the knee panel. The connection didn’t quite develop the predicted plastic moment, and it rotates to a considerable extent. Now, if this was the first plastic hinge to form, where you need on the order of 8 to 10 times the rotation of elastic limit, then the combinations aren’t bad. It almost develops the full strength after that rotation. However, if this were the last plastic hinge to form, there could be a resultant increase in deflection, that might be difficult if the deflection were critical. The connections with diagonal stiffeners weren’t more expensive to fabricate.

Dr. Newmark: What do you mean by this initial yield in shear shown in the figure.

Mr. Beedle: That is the calculated bending moment at which yielding, due to shear force, would commence in the knee panel. There are two. One assumes the shear is uniformly distributed. The other assumes the shear is parabolic.

Mr. Griffiths: I think it would be well to emphasize that even on the basis of ordinary elastic design as we do it now, you should generally have a diagonal stiffener...
FIGURE 1
DETAILS OF RIVETED TEST SPECIMEN

FIGURE 2
CUT-AWAY VIEW SHOWING METHOD OF GRIPING TEST SPECIMENS

FIGURE 3
TYPICAL FRACTURE SURFACES AT FOUR DIFFERENT SHEAR-TENSION RATIOS

FIGURE 4
NEWMARK
FIGURE 5

FIGURE 6

TABLE: SUMMARY OF TEST RESULTS

<table>
<thead>
<tr>
<th>SHEAR-TENSION RATIO</th>
<th>ULTIMATE STRESS BASED ON HOLE AREA, KSI</th>
<th>RATIO ULTIMATE STRESS TO RIVET TENSILE STRENGTH</th>
<th>RATIO ULTIMATE STRESS TO RIVET COUPON STRENGTH</th>
<th>RATIO ULTIMATE STRESS TO RIVET SHEAR STRENGTH</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0 : 0.0</td>
<td>44.89</td>
<td>0.796</td>
<td>0.725</td>
<td>1.0</td>
</tr>
<tr>
<td>1.0 : 0.577</td>
<td>45.54</td>
<td>0.787</td>
<td>0.719</td>
<td>1.015</td>
</tr>
<tr>
<td>0.577 : 1.0</td>
<td>53.92</td>
<td>0.907</td>
<td>0.847</td>
<td>1.201</td>
</tr>
<tr>
<td>0.0 : 1.0</td>
<td>59.47</td>
<td>1.0</td>
<td>1.044</td>
<td>1.324</td>
</tr>
</tbody>
</table>

FIGURE 8

NEWMARK
FIGURE 9

3/4" DIA DRILLED & REAMED HOLES

FAILURE
LINE

EFFICIENCIES

AISC RULE 75.0% 87.5% 93.8%

BY TEST 81.5 87.6 88.0

FIGURE 10

FIGURE 11

FIGURE 12

NEWMARK
SPECIMEN 50-SA  SPECIMEN 50-SB  SPECIMEN 50-SC  SPECIMEN 50-SA

---...

I

I

... +_·----_1-

[Image]  I" RIVETS...

7/8" RIVETS

~ 80t----+~___+_~___+__~-~-~-~-+-+-~--+-~--+---+--____I

>-

(D

...<-

5

IE

[Image]  3/4" RIVETS

20

7/8" RIVETS

... RIVETS

[Image]  RATIO OF BEARING STRESS TO NET TENSILE STRESS

FOUR GENERAL TYPES OF FAILURE OF TEST SPECIMENS

FIGURE 13

VARIATION OF "NET" EFFICIENCY BY TEST WITH THE BEARING RATIO

FIGURE 14

VARIATION OF EFFICIENCY BY TEST WITH THE BEARING RATIO

FIGURE 15

DETAILS OF THE SPECIMENS FOR STATIC TESTS

FIGURE 16.  NEWMARK
<table>
<thead>
<tr>
<th>JOINT DESCRIPTION</th>
<th>ULTIMATE SHEAR STRESS</th>
<th>ULTIMATE TENSILE STRESS</th>
<th>ULTIMATE TENSILE STRESS</th>
</tr>
</thead>
<tbody>
<tr>
<td>SERIES</td>
<td>S/T RATIO</td>
<td>JOINT TYPE</td>
<td>NUMBER OF BOLTS</td>
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<tr>
<td>S10</td>
<td>1.25</td>
<td>LAP</td>
<td>2</td>
</tr>
<tr>
<td>S11</td>
<td>1.25</td>
<td>LAP</td>
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<tr>
<td>S12</td>
<td>0.75</td>
<td>LAP</td>
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<td>S13</td>
<td>1.00</td>
<td>LAP</td>
<td>5</td>
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<td>1.25</td>
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<td>3</td>
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<td>S15</td>
<td>0.75</td>
<td>BUTT</td>
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<tr>
<td>S16</td>
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</tr>
<tr>
<td>S17</td>
<td>1.25</td>
<td>BUTT</td>
<td>2</td>
</tr>
</tbody>
</table>

**FIGURE 17**

<table>
<thead>
<tr>
<th>BOLT TENSION</th>
<th>PAYING SURFACE</th>
<th>PLATE FASTENER FAILURE</th>
<th>PLATE FASTENER FAILURE</th>
<th>PLATE FASTENER FAILURE</th>
</tr>
</thead>
<tbody>
<tr>
<td>KIPS</td>
<td></td>
<td>FAILURE</td>
<td>FAILURE</td>
<td>FAILURE</td>
</tr>
<tr>
<td>BOLTED JOINTS</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>M5</td>
<td>64.1</td>
<td>—</td>
<td>65.2</td>
</tr>
<tr>
<td>0</td>
<td>HQ</td>
<td>64.0</td>
<td>—</td>
<td>63.5</td>
</tr>
<tr>
<td>35</td>
<td>M5</td>
<td>65.1</td>
<td>—</td>
<td>66.1</td>
</tr>
<tr>
<td>35</td>
<td>HQ</td>
<td>64.6</td>
<td>—</td>
<td>66.0</td>
</tr>
<tr>
<td>AVERAGE</td>
<td>64.4</td>
<td>—</td>
<td>65.9</td>
<td>63.2</td>
</tr>
<tr>
<td>RIVETED JOINTS</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>M5</td>
<td>61.2</td>
<td>—</td>
<td>57.2</td>
<td>57.2</td>
</tr>
<tr>
<td>HQ</td>
<td>62.9</td>
<td>—</td>
<td>60.0</td>
<td>59.0</td>
</tr>
<tr>
<td>AVERAGE</td>
<td>62.0</td>
<td>—</td>
<td>58.6</td>
<td>58.1</td>
</tr>
</tbody>
</table>

**FIGURE 18**

**FIGURE 19**

NEWMARK
TABLE

<table>
<thead>
<tr>
<th>SERIES</th>
<th>VARIABLE</th>
<th>RANGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1, A2, A3, A4</td>
<td>BOLT TENSION</td>
<td>50% - 138% E.P.</td>
</tr>
<tr>
<td>A3, B3, C3, D3</td>
<td>BOLT DIAMETER</td>
<td>5/8&quot; - 1&quot;</td>
</tr>
<tr>
<td>A3, AG1, AG2, AG3</td>
<td>GRIP</td>
<td>1 2/3&quot; - 3 3/8&quot;</td>
</tr>
<tr>
<td>B3R &amp; B3</td>
<td>FASTENER TYPE</td>
<td>RIVET OR BOLT</td>
</tr>
</tbody>
</table>

FIGURE 20

RESULTS OF FATIGUE TESTS OF BUTT-TYPE JOINTS

<table>
<thead>
<tr>
<th>SERIES</th>
<th>STRESS CYCLE ON NET SECTION</th>
<th>STRESS CYCLES IN MILLIONS</th>
<th>AVERAGE FATIGUE STRENGTH AT 2,000,000 CYCLES</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>±18</td>
<td>1.21</td>
<td>17.1</td>
</tr>
<tr>
<td>A2</td>
<td>±18</td>
<td>2 ↑</td>
<td>18 ↑</td>
</tr>
<tr>
<td>A3</td>
<td>±18</td>
<td>2 ↑</td>
<td>18 ↑</td>
</tr>
<tr>
<td>A4</td>
<td>±18</td>
<td>↑</td>
<td>18 ↑</td>
</tr>
<tr>
<td>B3</td>
<td>±18</td>
<td>2 ↑</td>
<td>18 ↑</td>
</tr>
<tr>
<td>B3</td>
<td>±20</td>
<td>2 ↑</td>
<td>20 ↑</td>
</tr>
<tr>
<td>C3</td>
<td>±18</td>
<td>2 ↑</td>
<td>18 ↑</td>
</tr>
<tr>
<td>D3</td>
<td>±18</td>
<td>2 ↑</td>
<td>18 ↑</td>
</tr>
<tr>
<td>AG1</td>
<td>±18</td>
<td>2 ↑</td>
<td>18 ↑</td>
</tr>
<tr>
<td>AG2</td>
<td>±18</td>
<td>2 ↑</td>
<td>18 ↑</td>
</tr>
<tr>
<td>AG3</td>
<td>±18</td>
<td>2 ↑</td>
<td>18 ↑</td>
</tr>
<tr>
<td>B3R</td>
<td>±18</td>
<td>0.47</td>
<td>15.5</td>
</tr>
</tbody>
</table>

FIGURE 21

FIGURE 22

A COMPARISON OF THE SHAPES OF THE VARIOUS STRESS-STRAIN CURVES
FIGURE 23

Cycles to Failure
Carbon and Rimmed Steels

FIGURE 24

Cycles to Failure
Silicon Steels

FIGURE 25

Cycles to Failure
Low-Alloy Steels

FIGURE 26

Stress on Net-Section
Low-Alloy Steels

NEWMARK
in a knee like that. That is the way we build them at
the present time, unless special investigation is given
to the thickness of web inside that corner.

MR. BEEDLE: Yes, I should have mentioned that work-
ing stress of 20 ksi at this point.

MR. HIGGINS: Do you recall whether the web in that
square panel is of the thickness of a 14 inch member or
an 8 inch member?

MR. BEEDLE: Of the 14.

MR. HIGGINS: Probably that is about as bad a case
as we could encounter.

MR. BEEDLE: Yes.

CHAIRMAN JONES: Somebody else have a question?

MR. ENEY: Dr. Newmark, on the bolt test that you
showed at the University of Washington, the low value
of 23 per cent for co-efficient of friction, do you recall if
that used the hardened washer, and whether that was a
lubricated joint? That is asking for the recollection of a
lot of detail and it may not be possible.

DR. NEWMARK: I don't recall that because it was not
one of our own tests. On all of the tests that Hechtman
performed, however, he did use hardened washers. Some
had different treatments on surfaces, and this may have
been a painted surface. I don't know the answer. How-
ever, we have obtained values that low in some cases on
surfaces that have not been treated, but in which there
may have been some loose mill scale that may not have
been removed properly before the joint was put together.

MR. DERBY (Frank M. Weaver & Company): I have
a question for Dr. Newmark. Were all of those bolted
connections made with high tensile bolts and drilled
holes?

DR. NEWMARK: All of the bolted connections were
in accordance with the specifications which the Research
Council has suggested. All of them involved drilled
holes, with hardened washers, and high tensile bolts
meeting the ASTM specifications. In a few cases how-
ever the bolt tension was purposely low to investigate
the effect of this tension.

MR. BRINKMAN (Phoenix Bridge Co.): Professor
Newmark, in that relationship of rivet strength in tension
and shear, what about the initial tension in the rivets?
Was any account taken of that?

DR. NEWMARK: There was presumably no effect from
the initial tension, because the initial tension was lost
due to deformation of the rivet in the test. The initial
tensions could be measured. We could tell when the
discs began to separate that the initial tension was just
overcome. The values of initial tension of the rivets
ranged from about the yield point stress for the longer
grips, to something of the order of 20,000 psi for the
shorter grips. However, the failure in tension was at a
much higher stress, and the initial tension was completely
overcome before failure of the rivet occurred. This
would also be true in the cases where the rivets were
subject to combined tension and shear.

In the specimen subjected to shear, we could not
determine so precisely when the initial tension was over-
come. There was enough deformation, however, so that
we believe that all of the tensile forces were dissipated
because of the deformation that occurred. The rivets do
not very well yield in shear without permitting the
tensile load to drop off. We feel that this was the case,
because in the other series the results seem to be on a
smooth curve. There was no indication that there were
residual tensile stresses on the curve.

MR. ENEY: Mr. Beedle, I suppose we could think of
the floor beam in a single track long span through truss
bridge as approaching a simple supported design that
might have an application to this plastic hinge, having in
mind that the end rotation with even a heavy connection
to a hanger might initiate some of the hinge action before
we approach the yield point. Suppose that under over-
loading we develop first the hinges at the end, or perhaps
first at the center and then at the ends, would there be
any particular danger from overloading and working too
fast into the strain hardening range and therefore of
fatigue becoming a problem? Or am I going too far
into the unexplored region to make that a fair question?

MR. BEEDLE: Well, as I said, fatigue is a problem
which limits elastic design, and since it limits elastic
design it will to the same extent limit any use that is
made of the reserve strength in the plastic range. If
fatigue is really a problem, then we haven't invested
the fatigue problem at all. This work is directed primar-
ily to statically loaded structures, and since fatigue limits
the conventional elastic problem it would to the same
extent limit the use of plastic range.

MR. CHEW (Bethlehem Steel Co.): I want to ask a
question of Dr. Newmark.

On these tests that gave the failure of riveted joints,
the relationship of pitch to gage, could you summarize
your conclusions, as a guide in detailing a joint for a
tension member? What relationship between pitch and
gage would you recommend?

DR. NEWMARK: This is a little difficult for me to
answer just now. Roughly, the pitch should not exceed
the gage, and should not be less than some fraction which
I am not yet prepared to state since Mr. Schutz has not
completed his studies. He is still working on this prob-
lem. The spacing of the rivets should be fairly uniform
in the outer row, and the edge distance to the edge of
the plate should be at least half as great as the spacing
between adjacent rivets, and this should be of the order
of four and one half, or four, times the rivet diameter to
get the greatest efficiency.

However, other considerations enter and this is not the
only governing consideration. Mr. Schutz has made a
much more detailed study of other arrangements of rivets
where there is a staggering, where one row occurs midway between the other rows, and these results are still in a fairly complicated form. I am not prepared to summarize those because I don’t understand completely what his design rules are. He is now trying to put this into a simple form for his doctoral dissertation and we expect that it will be ready some time this summer.

CHAIRMAN JONES: It seems to me, looking at Schutz’s results, that first of all they are results simply on plates, rectangular plates connected in tension, and it is possible to make any relationship between pitch and gage and edge distance that you want to in those experiments, but when it comes to a box type tension member, where the angles to some extent control the edge distances, you get into another set of experiments, and they will be made.

In other words, there are variables ahead of us, so you can’t take these results directly into ordinary bridge design. We do have in some cases rectangular plates anchoring suspension bridges, maybe simple rectangular plates 50 or 60 feet long, and the efficiency of the end connection is rather important because there is that 50 or 60 feet on which you gauge your cross section on the efficiency of the connection.

It would seem from these results that our time-honored practice of advancing half the rivets a little way in front of the others and thereby figuring a greater efficiency is just on the illusory side. Instead of being able by advancing a few of the rivets to increase your apparent efficiency very much, it would be improved very little. If your tension member were short it would probably be just as well off to put your rivets in line and take out all of your holes. If your member is long, some kind of staggering in the first row might pay. Those are things that are going to come out of this research. What you have here is only a first phase.

Dr. Newmark has very well sold the importance of this work. I am trying to sell it a little further.

DR. NEWMARK: I think that one of the important results is that it is possible to get an efficiency greater than the 75 per cent which has been previously reported as a limit under certain conditions. Now these conditions may require expedients that are not practical. We are not yet prepared to say that we can achieve these with practical methods of design, but it is possible to get as much as 85 to 87 per cent efficiency. That is the upward limit. We feel that work must be done on elements other than plates, because we have no basis for extending this series of results to angles or other members.

If the results could be extended even roughly to such elements, it would indicate that some revision perhaps in the normal gage spacing on the connections should be considered. One should avoid holes close to an edge. It would be desirable to make the edge distance laterally as great as possible.

DR. WEBB (American Bridge Co.): May I ask Dr. Newmark a question? Referring to Figure 9, in the center, you have an efficiency of 87.6 and on the right 88.0. On the right the lower half is practically the same as the lower half of the center connection. What if you had the lower half of the connection on the right like the upper half? Would that change the efficiency?

DR. NEWMARK: The efficiency is determined by the rivets in the upper line and that spacing only.

DR. WEBB: The other half of the joints doesn’t count?

DR. NEWMARK: That doesn’t count.

DR. WEBB: Why?

DR. NEWMARK: We don’t know. Mr. Schutz has made a number of tests in which he has kept the specimen looking like the right hand one and arranged the lower rivets in a number of different patterns with practically no effect on the result. The distribution of stress on the plate on that outer section is what determines the strength of the plate, you see. There is a stress concentration or a strain concentration at the edge of the hole. We think we can explain the result on a semi-rational basis if the hole is close to an edge. The deformation in the plates causes this to open up very much and you get strain and tearing, but by interrupting the test before completion, we could show that the failure had occurred at the upper line and started out toward the edge and it hadn’t even started to crack at the lower line.

DR. WEBB: I am merely curious because the lower half of one is just like the lower half of the other.

DR. NEWMARK: Yes. This is a central plate of a double lap joint. The plates we are looking at are the outer plates. The failure is in the central plate. If the failure occurred in the outer plate, I think the results would still be the same. Some of the tests were designed for this purpose. In general, there would be slight variations of three to four per cent more in some of the other variables but these have a much smaller significance than just the one major variable.

MR. BLIX (Mississippi Valley Structural Steel Co.): Mr. Jackson, you referred to a steel that had considerably better properties than A-7 steel. Who makes this steel and what is the name of it?

MR. JACKSON: Well, it is now being manufactured by the United States Steel Corporation and they refer to it as Carliloy T-1 steel. There may be other similar steels, I don’t know.

CHAIRMAN JONES: I think if there are no further questions, we will thank the speakers for their able presentations and adjourn the meeting.

(The meeting recessed at five o’clock.)
The meeting reconvened at nine-twenty o'clock, Mr. Robert J. Wood presiding.

CHAIRMAN ROBERT J. WOOD: The meeting will please come to order. You might say that this part of the program is here by popular demand. I understand that the results of the questionnaire sent out after last year's meeting indicated that more of you were interested in hearing a panel discussion on welding than perhaps all of the other suggestions made put together. Consequently, this is your morning. I hope you will all feel free to join in the discussions that will follow the papers.

We have three speakers who will lead off the discussion. Between the three of them they have had a considerable amount of experience in all phases of structural welding, so we should cover the whole subject rather fully at the start and then open up for any questions and suggestions that you would like to add.

It was really amazing, considering how much welding has been done in the structural field in the past few years, to find that there are almost no undergraduate university courses taught in structural welding design, or, for that matter, in structural welding. It is amazing that there is a considerable lack of understanding, real understanding, of the fundamentals of welding by designers and builders alike, even after twenty-five to thirty years of structural welding. It is also amazing that there is such a difference of opinion, among designers and builders, as to what should be considered a proper, most economical method of connecting structural steel together by welding.

As a result, we find ourselves with different schools of thought and with almost no standardization. I feel that a discussion, an open discussion, such as I hope this will be, should be very worth-while.

Because of the nature of the opening remarks by our three speakers, I believe it would be well to hold all questions until the last speaker has finished, at which time you may address your questions or remarks to any of the speakers, or to the panel as a whole.

The first gentleman on our program this morning is Mr. Boyd S. Myers. He took his undergraduate and graduate work in civil engineering at Iowa State College. Later he taught structural engineering there for three years. Ever since then he has been in the general practice of structural design. For the last several years, he has been associated with the firm of Robert J. Cummins, Consulting Engineers, in Houston, Texas.

While his work has not been confined solely to design of steel, he has made a particular study of welded buildings and has designed many sizable all-welded structures which have been built in the State of Texas. Mr. Myers has also had considerable experience in the erection of steel, particularly by welding. He has written several papers on the subject of welding, one of which appears in the current issue of Civil Engineering, so I take great pleasure in introducing to you Mr. Boyd S. Myers.

Problems of Welded Fabrication—Part I
Panel Discussion
MR. BOYD S. MYERS

Mr. Moderator, Gentlemen of A.I.S.C.: In all structural welding the metal is subjected to very severe temperatures, temperatures of large magnitude. A 170 degree change in temperature in structural grade steel will cause a deformation of one one-thousandth of an inch per inch in the metal. The deformation of the same grade of steel at the yield point is approximately the same amount. Those statements give you some sort of an idea of the tremendous forces that are released by the structural welding heat that is applied to the metal.

This is illustrated very nicely in shop practice where it is necessary to camber a beam to meet the specifications, usually of the Highway Department.

Heat is always applied to the element which you wish to shorten, strange as it may seem. Consequently, that heat is applied to the bottom flange of the beam which, of course, will expand that bottom flange, but when that heat is dissipated completely, we find that the bottom
flange has been so shortened as to camber the beam at the center a given amount. The amount of that cambering will depend largely upon the amount of heat that is applied to the bottom flange.

The same thing is illustrated in the fabrication of columns in welded tier buildings. In the detail of columns practically all of the details are at the floor levels of the column. Consequently, heat is applied at those two points, and if the column is milled before the fabrication has been completed, you will find after the heat is completely dissipated, that the column has shrunk in length compared with the length before the fabrication was started. Therefore, it is desirable to do the milling after the fabrication of the column has been practically completed.

In field welding of multi-story buildings, the engineer is challenged by those powerful forces developed by the welding. There is no power known to man which will resist the movement of the metal either in shrinkage or in expansion caused by a large temperature change. The engineer’s problem then is to recognize those forces, visualize their action and develop that sequence of welding which will direct the use of these powerful forces for the greatest possible benefit to the structure.

Now, structural welding made considerable progress during our last war by its intensive and extensive application in the shipbuilding industry. As you will remember, there was a frantic effort to build cargo ships to overcome the destruction of the U-boats. There were large numbers of men employed, working twenty-four hours a day in three shifts, in all of the shipyards on the Atlantic Coast and the Gulf Region and Pacific Coast. Almost any workman who could hold a welding rod and do a passable job became a structural welder. When the war was over, there were a lot of men who had been trained in welding, but those men trained under war conditions, with the frantic speed that was required and without the check that we have in our civil activities, were found sadly wanting in their ability to deliver satisfactory welds.

Now, since the war, there has been a tremendous expansion in the construction of gas and oil pipe lines, and in the construction of pressure vessels for the petroleum and chemical industries. That expansion is still continuing today. There are approximately 10,000 miles of pipe lines in the picture for construction in 1952. About 6,500 miles of that will be oil pipe lines and about 3,500 miles of gas pipe lines. Today, the construction of pressure vessels and oil pipe lines is performed by welding. So, in this large expansion of welded work there has been a large number of excellent welders developed. Today, there are welders available, competent welders, for any job of a magnitude which requires their services.

There are two chief conditions that are retarding the application of structural welding as a general proposition. The first of these is the unwillingness of many fabricators to scrap good usable equipment and familiar methods of fabrication on a shop wide basis, and to purchase new welding equipment and spend the necessary cash to develop a good welding technique in their shops. No blame can be put on the fabricators for this. It is just a matter of business adjustment.

The second retardant to the use of welding is the persistent and almost unbelievable indifference on the part of designing engineers to the great possibilities inherent in good structural welding. Modern flame cutting and arc welding permits the wide awake designer to develop simple, effective details that were entirely impossible just a few years ago. Sizable savings in many types of construction, up to as much as 15 per cent, can be made by the employment of good structural welding.

The discussion later will show how, by intelligent direction of the sequence of welding, the magnitude and the kind of stress developed by the welding may be such that it will carry a portion of the floor load. This is an accomplishment that cannot be duplicated by riveted construction.

The type of details used, in connecting beams to columns or in splicing columns together, has a very definite influence upon the economy and the success of a welded job. On a twenty-four story building in Houston, one of the first built since the war, a column splice consisting of butt plates was employed. The original detail showed the column shafts milled before the butt plates were welded. It showed a second milling after the butt plate was welded in place. To do that would give you a column of exactly the correct length. However, to achieve economy, the fabricator expressed the conviction that the second milling was not necessary. Consequently, it was eliminated in the fabrication of this job, to our sorrow. In welding, that butt plate was badly distorted by the heat and cupped to some considerable amount so that after two or three such columns were set one upon the other, the elevation of column splices began to vary from what it should have been.

As those column splices varied up and down, they carried with them the spandrel beams. In this particular job, the spandrel angles were shop welded to brackets on the spandrel beams. Of necessity, the spandrel angles then formed a wavy line down the road. When the masonry was set it was necessary to cut loose most of those spandrel angles, re-adjust them, and re-weld them. It was a very expensive proposition.

With that sad experience from the butt plate column splice, it has never been used in our practice since.

The type of splice recommended is the old-time milled column section, with flange and web splice plates. When you stack a milled column upon a milled column, you can know definitely what the elevation of your column splices will be. In the usual case, the column splice is placed approximately at mid-story height in order to
square inches of butt weld, but if that flange is, say, 12 inches wide by a one and one-quarter inches thick, you would have 15 square inches of metal. It takes a lot more metal to make a 15 inch butt weld than it does a five. So, the total amount of heat thrown into the column from a butt weld will vary with the size of the beam flange.

Now, when a column is welded on one flange only, there will be a distortion of that flange from the heat. That will cause a movement of the free end of the column of some considerable amount in the direction away from the butt weld. But when that heat is dissipated, we get a shrinkage of that metal from the change in temperature and as with the other cases cited, that shrinkage is greater than the first movement by a considerable amount. Consequently, a greater force will be developed by the shrinkage forces than by the expansion forces. That seems to be true in all cases. I hope you will keep this in mind in the later discussion of connections of the beam to the column.

In some cases, where the welding is not under good control and the welding operators weld on only one flange of two or three columns at the same time, a force will be developed by shrinkage of the metal which will snap a three-quarter inch plumbing cable just like nobody's business. A tremendous force is brought to bear there. That is in the case where you weld on a single flange of a column.

This difficulty can be very easily overcome by utilizing your welding operators in groups of two, and welding on both flanges of a column at the same time. In that case, due to the expansion from the heat, you will get a very slight movement of the column upward at that panel point. But it will eliminate any possibility of a horizontal force being exerted at the floor above the point of welding which tends to throw the frame out of plumb.

If you will have the welding operators weld on opposite wall columns at the same time, you will experience those same forces but they will be approximately equal and opposite forces. When the weld is made, the forces will be outward in both cases. When the heat is completely dissipated, that force will turn in the other direction and will be inward, but both in an inward direction. As a result, there will be no particular force developed that will tend to throw your frame out of plumb.

The distortion of a building frame is entirely due to the effects of heat developed in the welding operation. The magnitude of the distortion is roughly proportional to the amount of heat that is applied at a given panel point in a given period of time. If a large amount of heat is applied at a panel point in a short period of time, it will cause much greater distortion than if the same total amount of heat is applied at three different intervals for instance, or over a long period of time, allowing the heat to dissipate itself.

Therefore, in order to limit the distortion of a build-
welding stresses. The amount of floor load that will be carried by these field welding stresses will vary somewhat, because of the fact that in the lower floors a larger proportion of the design is for carrying wind stresses. The amount of the floor load that is carried will vary from the lower floor to the top. In the upper floors it will be as much as 30 to 40 per cent. I hope that this discussion will inspire some of you younger fellows to make a research study of the stresses produced from this welding and report, later of course, to this group of gentlemen who are interested in those things.

So, the careful measurements that I have made verify the logical development of the stresses in the bottom flanges of those beams.

The modern office building requires very extensive electrical service of three kinds, namely, light, telephone and business machine connections. Even today practically all typewriters are electrically operated. The shifting of equipment for long-term tenants, and the shifting and moving in of new tenants makes this service a very difficult problem in most office buildings. Such service cannot be rendered adequately by any type of concrete floor construction, where the electrical conduits are placed either in the structural slab or in the floor fill, for the simple reason that you are limited to electrical outlets that you provide when the building is constructed. It is impossible to anticipate the needs for future tenants.

The best answer and the finest answer for this problem is the employment of cellular steel floors. This cellular floor is made near your city here, at Pittsburgh, Penna. As it is installed, a floor fill is placed over the top, two and one-half inches thick. This metal floor construction consists of hexagonal shaped continuous cells. Each of the cells is a wire race-way for electrical service. So, it is possible to get an electrical outlet anywhere on the floor using this type of floor construction. It may be either telephone, light, or business machines. Then, if you have a new tenant moving in and he says he wants an outlet here by his desk, he can have it. There is no other means, to my knowledge, that will provide that marvelous electrical flexibility for future tenants.

The floor, in a 16 gauge, weighs seven and four-tenths pounds per square foot. There are other types that are lighter and other types with thicker gauge metal that are heavier. The total dead load with this type of floor construction, exclusive of the weight of the structural steel itself, is always less than 40 pounds. Thus, there is a saving in dead load of 25 to 35 pounds per square foot of floor in the building.

Due to the reduction in the size of floor members, the columns and the foundations from this lighter floor load, and due to the much shorter period of construction, from three to five months depending on size and height of the building, this floor may be had by the owner without a penny of extra cost over what he would put into a building with concrete floors. Therefore, the architect or the designing engineer who ignores or refuses to consider this type of construction for office buildings on its merit is not giving his client the best service for his money. That is how strongly I believe in the cellular floor.

It might be important to observe here that the fill placed in this floor considerably increases its carrying capacity. This plain cellular steel sample floor section (sample shown) will carry 102 pounds per square foot on a 10-foot span with a deflection of one-third of an inch or 1/400th of the span. The same section on a 10-foot simple span with a floor fill will carry 415 pounds per square foot or approximately four times the carrying capacity of the plain section. This floor is furnished to the job usually in 25, 27, and 28 foot lengths. So, with continuity in the construction you can carry more load for a given deflection. It is safe to say that in ordinary use the floor will carry twice the tabulated loads.

The 16 gauge section I have shown you is listed for a 7-foot span at 380 pounds per square foot, for an 8-foot span 290, for a 9-foot span 250, for a 10-foot span 185. These loads are all based on simple span construction and on a stress of 16,000 pounds per square inch on the metal.

There are certain elements that are necessary to secure a satisfactory and desirable welded job:

The first is careful and accurate shop fabrication by a fabricator who has had experience in that class of construction.

The second is an erection contractor who has the proper equipment and the experience in erection of this type of construction.

The third is a field welding procedure which has been tried out and found satisfactory for the type of work involved.

The fourth is a reliable field inspection.

The fifth is carefully qualified welding operators.

The last two elements are probably as important as any of the others.

Field inspection should be placed in the hands of a recognized laboratory employing competent help. The qualification of welders should be in harmony with the standard qualification procedure recommended by the American Welding Society and should be carried out by an engineer representing the testing laboratory who shall be in personal, continuous, daily contact with the field welding operation. It is very important that the man who qualifies a welder is the man who is on the job to inspect the welding.

The effectiveness of the field welding procedure as recommended here is indicated by the following facts: On a building 252 feet long, 24 stories high, erected in Houston, Texas, the movement of the exterior columns due to the shrinkage of the welding was about one and
Problems of Welded Fabrication—Part II

Mr. W. L. Perry

Mr. Wood, Gentlemen: Mr. Myers has very adequately discussed the welded building frame from the viewpoint of the designing engineer.

In order to trace the development of this type of construction and point out the problems that the fabricator and erector have been faced with, I will briefly outline the history of the welded, steel frame building in the Texas Gulf Coast area.

In 1930 Mosher Steel Company fabricated and erected the Dallas Power & Light Company building in Dallas, Texas. This 18-story building, one of the largest welded buildings of its time, was essentially a building of riveted design with the rivets replaced by welds both in the shop and in the field.

Dr. Frank McKibben was largely instrumental in the design of the welded connections used for this building,
FIGURE 5

See Engineers Data for sizes of plates & holes.

TYPICAL COLUMN SPlice

TYPICAL BURD CONNECTIONS

PERRY
and you will find reference to the structure in his article in The Journal of the American Welding Society in the May and June, 1933, issues.

In Houston our first major structures of welded design were the Texas Company office building addition and the addition to the Houston Chamber of Commerce building. Both of these buildings were constructed in 1938, and each featured a different type of design.

From 1938 until 1946, there were no major welded tier structures constructed in the area. However, in this interval the welded mill-type building made quite a bit of progress. Dow Chemical Company, faced with corrosion problems due both to location and processes, built their Freeport plant of welded construction, featuring a design that eliminated inaccessible parts and allowed all butting surfaces to be sealed with a weld bead.

In 1946, the City National Bank Building was constructed in Houston. This 22-story building was built as a completely shop and field welded building. I mean completely literally. Built-up sections were used for both columns, and girders. It was all welded. The difficulties encountered in the construction of this building resulted in a search for a more economical type of construction.

Mr. Myers, working closely with the fabricator, erector, and inspector and aided by the change in allowable stress through butt welds, has evolved a type of structure that is economical to fabricate, erect, and one that will compare favorably in cost of the erected structure with similar structures designed, fabricated, and erected on a riveted basis.

The 1951 addition to the Gulf building in Houston illustrates Mr. Myers' latest design.

The Houston office building of the Prudential Insurance Company also built in 1951, and on which Mr. Ward Butterwick was structural engineer, used a type of connection for beam to column that is essentially the same as the connection developed by Mr. Myers. However, in many cases these connection details were modified to allow the use of negative moment reinforcement for the beams.

Before proceeding with a discussion of these buildings, I would like to point out that on the latest of them we have not omitted shop paint from surfaces that are to be field welded. We have found that where an iron oxide paint or a chromate paint is used for the shop coat, good welds can be made through this primer coat, and the welder is not harmed by the paint fumes.

I have had slides made from typical connection detail sheets for each of the five structures that I have mentioned. These slides are not as clear as they should be as they are made direct from detail drawings.

Figure 1 is a slide made from design details of the Dallas Power & Light Company building. It indicates the mass of detailed information shown on more than 40 similar sheets that were prepared before the details for the building were started. This building was built before the present day coated electrodes were in existence.

All welds were fillet welds. There were quite large fillet welds used both in the field and in the shop.

Many of the connections that were used on this structure are still being used by present day designers. In fact, I think Shedd in his book "Structural Design in Steel" devotes two or three pages to this building and the connections in it.

Figure 2 shows connection details used on the Texas Company's office building addition. The field connections were made by fillet welds that attached the beams either to seats or brackets which were shop riveted to the supporting column. Where seat connections were used, the top flange of the beam was not stayed laterally until the top flange was field welded. The connections with shop riveted seats in most of the cases had a loose angle field welded both to the beam and to the column after the job was erected.

Figure 3 indicates a type of wind bracket connection used on the Houston Chamber of Commerce building addition. The wind brackets were shop riveted to the column and were made from a split beam section. The spandrel girders were shop riveted, made of four angles and a plate, and were attached to the brackets in the field by fillet welds and an incomplete butt weld across the girder web.

These spandrel girders were encased in concrete and had spandrel angles attached to the concrete encasement by bent anchor bolts. The spandrel angles were set by the general contractor, and that got us away from troubles that we had on the City National Bank, at a later date, where this construction was not used.

Figure 4 shows how for certain bents the wind was carried across the building by field welded end connections on the beams. This is the first use of butt welds for this purpose in the Houston area that I am familiar with. The plates were widened at the butt weld in order to reduce the unit stress in the welds.

The top and bottom flange plates were shop welded to the beam and field welded to the column. Even if at the time this building was constructed, it was supposed that these plates should be left loose to allow adjustment in the field, we welded them on in the shop. The seats were shop riveted for shear. Both the top and bottom plates were attached to beams in the shop with fillet welds.

Figure 5 illustrates a typical detail connection used on the City National Bank Building. Note the column splice that Mr. Meyers referred to awhile ago. We did have trouble, lots of it. The floor got out as much as an inch and one-half in elevation between the different columns.

The beams did not frame with flush tops. You will notice, at Section A-A, that they are dropped an inch and one-half below the top of the girders.

The floor stringers, or intermediate floor beams, were
framed for continuity across the girders as shown. This proved to be an extremely expensive connection to make in the field. Actually, the beams over ran in depth and their flanges were cocked. We had some sliding allowances in the distance between top and bottom plates of these connections, but in the field the beams had to be driven into place. Entering the beams in the space allowed above the seat on the girder proved to be extremely difficult.

The allowable stress on a butt weld, at the time this structure was designed, was such that the full moment value of the beam could not be developed by butt welding the flanges directly to the column.

Plates of such widths and thickness as were required to develop the beam flange through a butt weld were shop welded to the beam top flanges where they framed to the column flange.

The cap plates of the seats upon which the beam rested were of cross-sectional area equal to the beam top flange reinforcing plates. These cap plates were shop butt welded to the column and attached to the beam flange by fillet welds in the field.

Connection of a beam web to a column flange depended on shear plates field welded to carry the end reaction.

The connection of a beam to a column web had no shear plate, and a stiffener under the seat carried the beam reaction. The top flange moment plate for beams framing to column webs were made in two pieces and field welded to both the beam and the column.

Figure 6 shows typical shop details of the spandrel beams on this structure. When the stonework was brought up to the first spandrel, we found what a mistake had been made when we fabricated these like this. The spandrel angles had to be cut loose in the field and re-located.

The spandrel angles were shop welded with a good bit of weld on each connection, and sent out as a unit with the beam, which was erected complete. As Mr. Myers told you, those spandrel angles looked like snake tracks across the face of the building. All of the buildings that have been discussed are good buildings and are adequately performing the services for which they were designed. However, the City National Bank Building was an education to all concerned. The education covered the field all the way from grammar school, through postgraduate study on what not to do on a welded building if economy is any object.

Figure 7 illustrates Mr. Myers' later design. This slide shows all of the typical details required for detailing the Gulf Building Addition, Number 2. In addition, it shows the field welding. All beams, framing to columns are welded for continuity. All beams are framed flush top. All beams not attaching to columns have standard shop riveted angle connections which are field bolted with ordinary machine bolts drawn tight with impact wrenches.

Spandrel beams are encased in concrete and the spandrel angles are attached to this concrete encasement by the general contractor through the use of concrete inserts. That, where it is possible, relieves us or any fabricator of quite a headache.

In Houston it is a pretty general custom to encase all of the spandrel beams and all of the exterior columns in concrete, even though the building may be built with a V-floor and the interior floor and beams themselves fire-proofed with vermiculite plaster.

Columns are spliced through milled ends and have both web and flange splice plates. The flange splice plates are attached in the shop to the upper column section. Web splice plates are shipped bolted to the upper column section. The splice is located five to five feet six inches above the tops of beams as a maximum because of the erection problem when it is located higher. We put the column splice plates on the upper section of the column in order that the heavy weld necessary to take care of the fills can be made in the shop.

Beams framing to column flange are beveled for down hand butt welding to the column flange. The webs of these beams are cut back approximately one inch from the face of the column flange. The end reactions of these beams are carried to the column through a shear plate shop welded to the columns and fillet welded in the field to the beam. Each shear plate and corresponding beam web is provided with a minimum of three erection bolt holes. A 4 x 4 angle erection seat is shop welded to the column, one-quarter of an inch below the theoretical bottom of the beam.

The flanges of beams framing to a column web are beveled for down hand butt welding to the flanges of a synthetic beam section. This synthetic beam section is shop welded to the column and is made up of column web stiffeners located opposite the top and bottom flanges of the attaching beam and a vertical shear plate that laps the beam web. The beam web and vertical shear plate of the synthetic beam are joined in the field by a fillet weld. Erection bolt holes are provided just as for beams framing to column flange. An erection seat is also provided and it is located one-quarter inch below the theoretical bottom of the beam.

The plate marked W is the web plate of the synthetic beam. It is nothing more than a plate of the same thickness or approximate thicknesses as the web of the beam that frames to it. The column stiffener plates Pt are welded into the columns either by full penetration butt weld, or sometimes by fillets across the web. If they are attached by fillets, they are always attached to the flanges by butt welds. The seat plate, that we provide on the bottom of the synthetic beam, is bolted on through a
fill to drop it a quarter of an inch. We put two bolts through it and it does the job.

The only difference between the end of a beam framing to a column flange and one framing to a column web is the manner in which the beam web is cut.

The width of the column web stiffeners that form the top and bottom flange of the synthetic beam section are held to a constant figure from the face of the column web, not from the center line of the column. This causes the length of the beams to vary each time the column weight is changed. This distance is set in round figures as an inch and one-half to two inches beyond the toe of the flange. This figure is sometimes changed when the nominal flange width of column changes but by preference should be constant for an entire structure.

The bevels for the flanges of the beams are cut on a 45-degree angle, and are brought down to a feather edge. The beams are detailed and fabricated to a length of plus O-minus 1/16 of an inch of the theoretical clear distance between column flanges or edges of synthetic beam sections. No allowance is made for overrun of columns.

All stiffener plates, synthetic beam web plates, and so forth, are cut to size on a burning table by the use of a mechanical torch. Where column web plates require beveling for butt welds, these bevels are cut by a tractor type torch travelling on a track.

Plates after edge preparation are tack welded in place in the column, their location checked and passed by the shop inspector, and then while on the skids, all fillet welds are made. Next, the columns are hoisted into a vertical position and the butt welds connecting the column stiffener plates to the column are made.

The lengths of these columns will not be the same after welding is completed as they were before welding began. Once recognized, this will not seriously disturb the shop, for the shop is forced to juggle splice lines on riveted columns to take care of under run, in length, of columns ordered from the mill, and so will already have a procedure worked out that will cover this contingency. It probably will be a procedure that will eliminate milling these columns after welding is completed.

Beams, framing to columns, have their flanges beveled by tractor type torches running on tracks that rest on the beam flange which is to be beveled. Their webs do not have to be accurately cut to line and are, therefore, cut by hand torches.

Holes for these beams are in the web so that they can be punched and do not require drilling, as might be the case if fit up holes were through their flanges.

As for erection, a butt weld will shrink—we all know that—and for this reason, a structure of this type cannot be plumbed throughout a tier before welding is started.

Our erector has found that by plumbing his elevator shaft columns and welding the beams to these columns, then by spreading the adjoining columns out the amount of the expected shrinkage and welding the beams to these columns, and then proceeding to the next columns after checking his actual shrinkage on the first bay welded, that he can hold his building to size and plumbness as close as any riveted building can be held.

The feather edge cut on the beam flanges will ordinarily have a gap between the end of the beam and the column due to spreading of the columns to allow for expected shrinkage. Since back up plates are used for all field butt welds, this will form a satisfactory joint. Where due to overrun of column the feather edge is hard up against the column, the welder will have no difficulty in obtaining full penetration. Where as occasionally happens the beam must be shortened slightly to adjust the bay width, the feather edge is burned off as required, but bevel is still maintained.

This adjusting of bay widths to allow for shrinkage does not affect the beam web connection since the web is connected by fillet welds to a lapped plate.

The erection seats provided on the column allow these adjustments to be made. Since all beams have erection bolts spaced throughout their webs, there is no tendency for the beams to rock under erection loads and the erector has a stable floor system from which he can work.

Figure 8 shows typical details for the Prudential Insurance Co. building. Note the similarity to the beam and column detail, just discussed. However, where negative moment plates are used on the beams, the corresponding column web stiffener plates were divided into two plates. This division of the plates saved quite a bit over one plate of the same combined thickness. Section A-A of Interior Columns shows the negative moment plate extending various distances out along the beam.

While the typical detail shows a 45 degree bevel on the beam flange extending on through the flange cover plate, it was found that a modified U-groove weld could be obtained by squaring up the edge of the cover plate and placing this square edge at the edge of the bevel on the beam flange. A similar U-groove was used at the bottom flange. Here the flange was square and the cover beveled.

This was the first structure that we fabricated where beams were framed for continuity to columns having cover plates, the beams framing into the covers themselves.

We were given an opportunity to try several methods of transferring the stress from the beam flange through the column cover plate to the column itself, and found, by experiments, backed up by tension tests, that we could accomplish this through the use of plug welds made through one and a quarter inch diameter holes spaced as shown on the following slide.

Figure 9 is a schedule for these plug welds. The holes are placed with very little distance between their edges and are spaced in such a manner that we can develop a
beam flange through the column cover plates on the consideration that the holes are rods, stressed at 20,000 pounds per square inch. Actually, this was very, very conservative because the weld penetration was such that it was almost equivalent to a solid weld across the face—not quite but almost.

This plug welding was limited to plates of one and three-quarter inch maximum thickness, and were made by an automatic submerged welding machine using five-sixteenths electrodes with 1800 amps. and approximately 35 volts. The welds were made by time—a stop watch being used to start and stop the welding—with time per hole approximately one-half minute. Mr. D. C. Ransome and Mr. N. G. Schreiner of Linde Air Products Co. assisted in the experiments leading to the adoption of these plug welds.

We also experimented with slot welds two inches wide and about twelve inches long, made by this same automatic welding machine using a backward and forward motion without interruption of current, but the slot welds entrapped slag and were not considered satisfactory. We did, however, use two inch wide slot welds through two and a half inch thick plate where the welds were made by hand welding with A.C. Machines. These welds proved acceptable.

The building, faced with marble, had spandrel angles that hung from the bottom of the spandrel beams. This standard detail sheet, Figure 10, shows how these angles were bolted through slotted holes to allow for field adjustment. The spandrel angles were assembled to the spandrel beams in the shop and were erected as a unit with the beams. No attempt was made to line up the spandrel angles until the marble was almost up to them. In fact, I think the erector worked from the same scaffold as the masons did. This proved very satisfactory from the general contractor’s viewpoint, but involved the erector’s keeping a crew of men on the job long after the rest of the steel was erected. Needless to say, this was very expensive.

These latest welded buildings, as typified by the Gulf Building Addition and the Prudential Building, make use of welding, riveting and bolting, each used in its appropriate place, and you, as fabricators, can see why we like this construction.

That concludes my remarks. Thank you.

(Editors Note: Mr. Perry also showed a series of 25 color slides to illustrate the appearance of the structures, and their details, as discussed in his paper.)

Chairman Wood: Thank you very much, Mr. Perry. We surely enjoyed that. I am sure that you will have a lot of questions asked of you.

Our next speaker is Mr. J. R. Stitt. Mr. Stitt graduated from Penn State. After studying in this field, he worked with welding first both as welder and sales engineer. Later, in 1938, he was instrumental in establishing at Ohio University what I understand was the first and perhaps even today the only Department of Welding Engineering at a University. He taught various courses for a few years thereafter in that department.

Mr. Stitt is now the research and welding engineer for R. C. Mahon Company in Detroit. He has had considerable shop and field experience in structural welding, and has been for several years active and prominent in the American Welding Society. He is an authority on sequence of welding to control distortion, and on straightening by the application of heat.

We will be happy to hear from Mr. Stitt.

Problems of Welded Fabrication—Part III

MR. J. R. STITT

Mr. Chairman, Gentlemen: Mr. Myers has talked to you about design. Mr. Perry has talked to you about fabrication. I am here to straighten out a few things, I mean a few beams, and so forth. We will move right along so that there will be plenty of time for discussion of all three papers.

Apparently I haven’t had my postgraduate degree in working on tall buildings, 16, 18, 24 stories high, but I think I can fill in some gaps by showing how we have overcome a number of difficulties which occur from shrinkage. Shrinkage often causes distortion. Both Mr. Myers and Mr. Perry have mentioned the spandrel beams that looked like snake tracks, and the difficulty that occurred on the column splices which were not milled after welding.

There are certain things that can be done to greatly help these two situations and I will point those out to you from a number of slides that I have.

If you will learn to control shrinkage, your welding problems in fabrication and erection of structural steel will shrink. They will shrink to a minimum. If you will learn to control shrinkage, your welded structural members as well as the completed structure will have the correct dimensions.

Mr. Perry mentioned how the columns had to be shoved out in order that the building would shrink back to the proper size. That is often done and it is a very effective way of doing it. Shrinkage occurs wherever welding occurs. A piece is always shorter, both in length and width, wherever there is any heat applied to the
beam, and there has to be heat applied to it in electric arc welding, in resistance welding, and in most types of welding except the so-called cold welding. One way to have the piece end up the right length, is to mill a column after welding, with due allowance being made for shrinkage and milling. Beam lengths can be controlled by trimming the beam afterwards, if it is a beam or box section of some kind.

In ship construction they have the same difficulty, because they have a great deal of welding on the length and a ship 5 or 6 hundred feet in length could end not just inches short, but feet short of the original planned length. That is overcome by allowing for the shrinkage, and after the shrinkage has occurred, making final joints which are trimmed to suit. I don’t believe that on a tall building anyone is going to measure and find that their building is a few inches short of the planned 24 stories high, but on a ship, if the customer receives his ship and it is, let us say, 18 inches short of the planned 500 feet, he will claim that he is short-changed a number of cubic feet, and, therefore, a great deal of tonnage, of cargo, because that 18 inches will be right out of the middle of the ship where you have the greatest cross-section. The width and height also is changed and it is often necessary to build in some allowance for shrinkage in the original piece to overcome these.

Your welded structural members will also distort. That is because more shrinkage occurs on one side than on the other from the welding.

The other advantage of understanding shrinkage is that you will be able to straighten distorted members, such as the spandrel beams that were mentioned in the multiple story buildings.

I plan to discuss these three items in just the reverse order, because I believe that if you understand how to straighten a distorted member, you will know how to weld a member and keep it straight.

I have been experimenting with straightening and have taken a flat bar, Figure 1. The bar was 192 inches long. It was two inches wide, and a quarter of an inch thick, and by applying a series of heats, as shown by Heat Series No. 1, all the way along the bar spaced approximately 8 inches apart, that bar was bent the stiff way to the shape shown. At Series 1, it had a 41\(\frac{3}{16}\) inches offset at the center. Next, applying the heat at Series 2, in between the original heats, we obtained the position shown at Series No. 2. We had an offset of 9\(\frac{7}{8}\) inches, or just about double the original amount. The third series of heats were applied between 1 and 2, and the bar curved some more up to 141\(\frac{3}{4}\) inches. After 6 series of heats had been applied, there was an offset of 26\(\frac{1}{2}\) inches.

Carrying it further, Figure 2, 12 series of heats had formed it around to a shape with an offset of 481\(\frac{1}{16}\) inches, and at the end of 18 series of heats, it had a central angle of 178 degrees, almost a half circle, at which time its length of 192\(\frac{7}{16}\) inches had changed because, on the inside of the curve, we had 188 inches. What happened to the other 41\(\frac{3}{4}\) inches? The metal on the inside was thicker than it was originally. We didn’t throw any metal away. We merely changed its position or shape. The outside had elongated 2\(\frac{1}{4}\) inches. In other words, it was thinner and had stretched.

Now, from this position, that bar could be straightened back to its original straightness as shown at the top, at which time the piece would be approximately one inch shorter than it was originally. There is no such thing as saturating that bar with stress so that it cannot be bent or straightened by applying additional heat to it.

Figure 3 shows a 2\(\frac{1}{4}\)-inch WF beam. If you will follow the left hand edge of the top flange up to a point where it starts to bend, and continue, you will be on the opposite side of the beam.

Figure 4 shows that beam straightened, straight as a string, and the top flange is about an inch thick. So, you see that heavy members can be straightened. Much heavier members can be straightened if you can apply the heat quickly enough.

Since we find that the heat will deform members, we must control it while we are welding so that we keep our members straight. I have pictured in Figure 5 a large I-beam, being made out of three plates, just for something to talk about, so that I can show you how I would apply the weld to that member to hold it straight regardless of its length. It can be held straight by sequence welding. First the beam is placed in Position 1, welding fillet welds at Flange B. My system uses one-half of the total weld that is to be applied at this point. If it is a multiple-pass weld, the first layer can be applied and then the beam rolled over 180 degrees to Position 2, putting the first layer of welds at Flange A. If it is a single-pass weld, single-pass fillet on each side, welds can be made of a length that can be deposited to that size by one electrode, and then a space left unwelded and another weld applied, and so on, leaving half the amount of welding required at the time that it is rolled over to Position 2. Then the same amount of weld can be applied to Flange A. Without changing its position, the weld should be completed against that flange Position 3. Then it is rolled another 180 degrees, putting Flange A again at the top, Position 4, and the original weld against Flange B can be completed.

Now, what shall be do about the welds being on two sides of the web plate? If you have two operators available or want to weld it automatically and have automatic equipment available, that’s fine. If you don’t, one welder can weld on one side and then on the other side, back and forth, and it isn’t particular whether he welds from one end to the other or starts in the middle and welds towards the ends, or starts at the ends and welds toward...
the middle. It makes little difference, as long as you keep the weld relatively balanced on the two sides.

It works out this way, that when the weld is made at Flange B, Position 1, there is a shrinkage occurring not only in the weld but in the metal adjacent to the weld, and there is several times the volume of base metal that you have in volume of weld metal that is shrinking along with the weld metal causing the distortion. That will shorten Flange B, which will give you a camber in the beam. If you were to complete the weld on Flange B, and then turn it over and complete the weld again on Flange A, Position 2, your beam would not be straight. Why? Because, as the shrinkage occurs at Flange B, the beam is cambered and it will resist the shrinkage of Flange A from pulling it back straight.

All we have done at Position 1 is to do a portion of the weld, roughly 50 per cent, turn it over, Position 2, and shrink the opposite side, try to shrink it the same amount. It won't shrink quite as much, therefore, the beam at this point is not straight, but by applying more heat to Flange A, Position 3, you can pull it back a little past straight, and then we complete it by doing the last welding, Position 4. We will pull it back straight.

The same system can be used whether it is a box section with four welds at the corners, or whether it has a butt weld, or double V-butt weld welded from the two sides. The same system works. Now in Figure 6, Weld 1, Position 1, will fuse and give a heat affected zone plus the additional fused base metal and will tend to shrink at right angles to the axis of the weld, as well as lengthwise. By turning it over to Position 2, and applying Weld 2 of the same size as we have laid down before, it will tend to pull across the axis of the butt weld as well as lengthwise and it will slightly distort the plate. Placing Weld 3, Position 3, will pull that plate past straight, and when you turn it over and weld the last weld, Position 4, you will have the piece end up flat.

I have seen beautiful colored slides and also movies of welding sequences, where one manufacturer of welding equipment and electrodes, shows the first weld deposited, turning it over and making the second weld opposite, turning it over again to put a weld above the original weld, and then turning it another time to complete the welding. I claim that this will not end up with a straight piece. It is an extra turning operation and it won't come out as straight as with the sequence shown.

We have built a number of structures where we have applied this. One that was very complicated, because of diaphragms placed at intervals along its length was a Taintor Gate dam, 90-feet long. It had to be good and straight, because there is a hinge along the top of the 90-foot length of the dam, and it had to rotate. You need a good, straight section to rotate. By balancing the weld,—not putting all the welding on one side before balancing it with some welding on the other side,—we were able to keep that piece very straight.

We have also had sections such as the stiff-leg on a large power shovel. One happened to be about 55 feet long and only 21 inches wide, and we held those members to within an eighth of an inch by controlling the sequence.

There are other things that you can do on structures of this type to help yourself. One is to make sub-assemblies, and then fit those together, and again control the sequence as the sub-assemblies are fitted together and welded.

Mr. Jackson yesterday in his talk, mentioned that stretching a piece of steel up to the yield point changes its length one-thousandth of an inch per inch. One of the speakers this morning mentioned that 170 degrees Fahrenheit change in temperature is all that is necessary to produce this same change of length, one-thousandth of an inch per inch. I worked it out mathematically and have also experimentally worked it out to prove this point.

Figure 7 shows you my reasoning. I have cut two openings in there, leaving the cross-hatched areas that represent one square inch of cross-section at the middle B, a half of a square inch of cross-section at A (left), and another half of a square inch at A (right). Now, if I could apply heat to the mass of steel A+B leaving the remaining mass of the specimen cold—room temperature—the metal Mass A+B wants to expand, and become longer. If I cut that metal Mass B out, and raise Mass A+B to 170 degrees above room temperature, the specimen will elongate one-thousandth of an inch. But if I leave it in the structure and apply the heat to those two sections, the specimen cannot expand freely that one-thousandth of an inch because Mass B forms a tie across, which also has one square inch of cross-section, and which is going to try to maintain one-inch distance between the two end beams. So, when heating A+B to 170 degrees above room temperature, the specimen tries to expand one-thousandth of an inch, but it will only expand five ten-thousandths of an inch, a half of the amount, because it has to stretch Mass B along with it, assuming that the beams at the ends are rigid enough so that we can neglect deflection. A and B expand due to the thermal change; going into the compression, and B is strained and goes into tension and elongates the same amount. At 170 degrees we will have one-half of yield point compression in A and A, and one-half of yield point tension in the center tie B. So, I apply another 170 degrees to it, a total of 340 degrees, and at that time I have yield point compression and yield point tension.

Now, as I apply another degree of temperature, something has to yield. So, why did we say 170 degrees is all that is necessary? Here, I now say we have to use 340
degrees. Well, 340 degrees is necessary in this specimen because our area that is being heated is held back by an equal area that is cool.

Let's look at the next slide, Figure 8. Now I have one-tenth of a square inch cross-section at C. Now, when I apply 170 degrees to A and A, if the tie C were cut, it would go out the full one-thousandth of an inch, but it is held back by tie C. In this specimen the tie area is a tenth of a square inch, and so, it is only necessary for me to heat the outside masses A and A to eleven-tenths of 170 degrees, or 187 degrees, to stress the metal at C right up to yield point.

Since we must have equal and opposite reactions, the compression is going to be quite low because we have one square inch carrying the compression, and we have one-tenth of a square inch carrying the tension.

Let's just assume that we can machine the center tie on down so we have just a very thin ribbon of steel in between the end beams, one-hundredth of an inch wide. By applying just slightly over 187 degrees at A and A, we will put C right up to the yield point. Now, if we apply any more heat, we will stretch that metal. Let's put 250 degrees on it, for instance. Then C will have to stretch along with it because we have a small area. All right then, we let it cool. We just walk away from it. The Mass C metal will go into compression as the outside cools down because it already has taken a permanent set. It has practically deformed, and it will go into a compression upon the cooling of the outside members A and A.

I know I have taken quite a little time on these two slides, but it is very important that we realize how little temperature change is necessary to produce these distortions.

The next two slides will show an experiment that was run to prove this point, and how stresses can be built up in members by thermal change. In Figure 9, the section at left is a solid piece about 2½ inches in diameter to which two collars have been welded, a solid shaft with two collars on it. At right is a piece of pipe which was slipped over the top so that it over-lapped one-half of an inch on each collar.

Figure 10 shows the piece as it was welded to the collar. For the time being, just forget that the picture shows the sleeve cut in two at the bottom because that is the next operation. So, it was welded to the collars which, from the shrinkage of the weld metal alone, would tend to shorten the tube putting tension in the tube and compression in the original round shaft. I wasn't concerned about how much stress was set up from that welding, because I put the specimen in a stress-relieving furnace and heated it to 1200 degrees Fahrenheit, a good red heat, and that wiped out 90 per cent of all the stress that had been set up by the welding. But rather than leave it to cool slowly in the furnace, the piece was set out in still air to cool. Thermocouples were placed in four different positions in this specimen. One was on the surface of the tube. One was near the end of the tube. One was imbedded in the shaft at the center, and another one was inside of a drill hole at a point right about under one of the collars. The thermo-couple readings were made as the piece cooled in still air. The spot that cooled fastest was on the outside of the tube. The next was at the end of the tube. The next was in the drill hole down inside the shaft under the collar, and the last one to cool was in the middle of the shaft. Naturally, it was shielded from any current of air by the outside tube. When the piece was cooled down, we found considerable stress set up by the thermal differences as it cooled.

Very accurate strain gage readings were taken on the outside tube and from collar to collar. Then we cut through the tube and relieved the stress that had been set up in both the tube and in the shaft, measuring the change of length that occurred. The tube elongated. It was in compression from the thermal cycle that it had been through. The shaft cooled enough slower than the outside tube, so that the tube, in cooling down, had taken a permanent set, had stretched. Then, when the center shaft finally cooled down, the tube was too long, so it was stressed in compression. Being in compression, as soon as we cut through it the tube elongated. The measurement of change of length in the 8-inch gauge length indicated 22,600 pounds per square inch were in that tube at the time that it had cooled to room temperature, more than you allow on usual design stresses.

The shaft had five times the cross-sectional area of the tube. We set the stage that way by selecting a tube of a thickness which would give us one-fifth of the cross-section area. A measurement from collar to collar on 10-inch gauge lengths indicated that the shaft had 4,600 pounds per square inch tension stress. In other words, it is a reverse—the smaller area has five times the stress.

I am now going on with problems that arise from welding. We have recently completed a viaduct in Cincinnati about three-quarters of a mile long. There were some 187 columns under the structure. There were a variety of columns under the different ramps and approaches to the main viaduct.

Some of this was designed as a rigid frame, and therefore the columns were supported on a ball and socket joint made up of a lower shoe plate with a heavy dome contoured top piece welded to it, and an upper shoe plate with a heavy cup piece welded to it.

Figure 11 shows the arrangement. The socket piece was welded to the upper shoe with a heavy fillet weld, and the dome section was similarly welded to the lower shoe. The shoe base plates varied in thickness from 1 inch to 4½ inches, depending upon the particular load that was on the column. Now, in welding the thinner
FIGURE 3

FIGURE 4

WELDING SEQUENCE

1
FLANGE 'A'

2
FLANGE 'B'

3
FLANGE 'B'

4
FLANGE 'A'

FIGURE 5

FIGURE 6

STITT
Figure 7: Mass $A + A = Mass B$

Figure 8: Mass $A + A = 10 \cdot Mass C$

Figure 9

Figure 10
**WELDED COLUMN BASE PLATE**

**FIGURE 11**

**Sketch A**

**Sketch B**

**Sketch C**

SECTION THRU LOWER SHOE

**FIGURE 12**

PREFORM BEFORE WELDING BY CLAMPING TO STRONGBACK

**SECTION A-A**

**FIGURE 13**

PREFORM BEFORE FLAME CUTTING & WELDING

**FIGURE 14**

WELD IN PAIRS BY DEFORMING & CLAMPING OVER BLOCKS

**SECTION A-A**

REQUIRED FINISHED WALKWAY CHANNEL

**STITT**
that point. But, in general, our erecting contractors raise no objection. From the standpoint of cost, I don't believe it is of material consequence and that, in the final analysis, is the answer—the cost.

MR. CORBIT: I would like to ask Mr. Stitt a question. Frequently, base plates welded to columns cup—practically all the time, I should say. I was wondering if the use of the copper wire at the bottom of the column might relieve this. Do you suppose that could be practically used?

MR. STITT: On the large power shovel, we had a box of high tensile steel along both sides of the stiff-leg, 21 inches square, with diaphragms up and down through the box every three feet or so. We applied the compressible spacers, as I like to call them, as we fit the four plates around the diaphragm.

The same thing applies; we have a diaphragm and put a compressible spacer in there before the fillet welds are applied. This will shrink and will have a minimum of distortion.

By putting the copper wire in there, the two fillet welds can shrink without having a point to pivot around to dish or pull the diaphragm and side plates. Your plates on the sides, top and bottom, on such a box section will be very flat. If you don't do that, you will have a scale opening effect, where you can pick out every diaphragm from the outside just by looking at the shadows down along the piece. It is very helpful, and it doesn't take very long to put these copper wires in. The fitters don't like it, but it helps a lot.

MR. CORBIT: What about the column?

MR. STITT: On the column base plate, the same would apply. The shrinkage would be in one direction. You do have the additional effect of relieving stresses from the plates, that were rolled in at the mill. Such a plate has a tension in its mid-thickness from the cooling and compression on each surface. The fillet welds applied on one surface are going to relieve that compression because you melt the surface which is going to tend to pull up some from that.

Now, the question hasn't been asked, does that copper squeeze down, and it definitely does. Just a copper foil is left in there as the thing squeezes down. You don't have to worry about not having bearing because of the tiny copper wire there.

General Electric Company found that by machining a surface to put very fine points on its face—giving it a rough machine, so to speak—that this separated the parts a little bit, so the weld shrinkage squeezes those points down. They actually have a symbol that they put on their own company drawings that tells the machine shop not to machine a surface flat, but to leave the tiny points up.

MR. HALL: On comparatively heavy sections, would you recommend just running copper wire in the center of the piece or would you put one adjacent or close to each fillet weld?

MR. STITT: On a heavy piece, you can lay it right along each of the fillet welds.

Another study I made was in making just mere tack welds. I have put 14,000 pounds per square inch into certain assemblies, measured with strain gauges, just by tacking it in certain assembly sequences. If you put the compressible spacers in there, you can practically wipe that out.

CHAIRMAN WOOD: In answering Mr. Corbit's question, would this be a good method of keeping base plates on columns? Would the spacers be used on flanges, and web both?

MR. STITT: I would definitely put the spacers in there to hold the plate flat.

MR. CORBIT: The H-section is placed on skids on one flange, and the plate normally is tacked in a vertical position, so there would be a little mechanical problem of holding the wire.

MR. STITT: Yes, it is a little problem to hold the wire, but the wire can be brought over the top and down over the member, letting it dangle out below.

MR. CORBIT: I have another question. Do you have any methods that you have arrived at for computing how much of a space to leave between certain size beams for certain effects, or is it all cut and try?

MR. STITT: It is pretty much cut and try, because unless you have had that same section and the same amount of welding to contend with before, there is no way of calculating the effect that I know of. It is not a science at the present time. The same is true with straightening. It is a matter of knowing where to apply the heat, how much heat to apply, and if you apply the heat the same way every time, you get the same results.

MR. FOEHL: In the matter of this copper wire again, about how far back from the weld do you have to keep it to keep the copper wire from melting? Isn't it a rather low heat that will melt copper wire?

MR. STITT: Well, your copper wire will melt, but the only thing is to separate the pieces. Now, in the base plates, where we had the dome, say the piece is 9 inches in diameter, just make a ring of copper wire 6 inches in diameter.

MR. FOEHL: In the matter of the H-beam, where you might have a flange only 3/8 thick, you are not going to be able to get the copper wire far from the weld.

MR. STITT: All right, your copper wire is going to be heated, and it is going to soften.

MR. FOEHL: It wouldn't melt first?

MR. STITT: No, and you can put a lot of copper wire in and it won't affect your weld. That percentage of copper isn't going to bother any welding. A lot of gas welding rods are coated with thin films of copper to keep from rusting and a certain amount is good for the weld.
Mr. Black (Robberson Steel Co.): Mr. Perry, you mentioned about paints, that you use iron oxide or zinc chromate, where the field connections occur. I wonder if you have had any experiments with red lead or any other paints?

Mr. Perry: No, sir. I understand that there have been some experiments made with red lead, but we have not seen them and, frankly, I don't think we can get welders in our country to weld over red lead, whether it is dry or not. I understand if it is dry and in a thin film, it will work all right, but I would be afraid, and I don't think you will get many welders to do it.

Mr. Ball (Bethlehem Steel Co.): Mr. Myers, in connection with the synthetic beam welded to the web of the column, that welding was done in the shop, I understand. Now, do you make the flange plates thicker to take care of the overrun or underrun?

Mr. Myers: It is usually specified to the nearest eighth of an inch, of flange thickness. In no case will it be slightly thicker than beam flange. We specify that nothing thinner should be used, than to the nearest eighth of an inch.

Mr. Storer (Steel, Inc.): Mr. Stitt, in your first slide, you had a beam, if I remember, that had a pretty healthy bend. Your next slide showed it completely straightened. My question is this: When you straighten out a beam with that much bend, you are obliged to set up internal stresses of considerable magnitude. How far can you go in all this straightening process by the application of heat before you have to worry about what happens inside the beam rather than just the appearance?

Mr. Stitt: That beam can be rolled around into a circle, if you want, the same as the bar was. There is absolutely no more locked-up stress when I finish straightening or deforming a beam than there was in the original rolled section. I can take a beam or a bar, any regular rolled section, or I can take built-up sections, and change the shape of those by applying the heat without introducing additional stresses.

It may be hard to believe, but let's take a rolled beam as received. I can apply heat and deform that. I can turn around after it has cooled down, and apply heat in the opposite direction and bring it back straight. The same amount of heat will bring it back straight.

Now, if we have put new stresses in there, the second application of heat would not have brought it back straight. I have done that many times. In fact, last Friday, I gave a demonstration in Grand Rapids, Michigan, for the American Welding Society, where I deformed beams, and brought them right back straight with the second application of heat.

Mr. Puttsche (Braden Steel Corp.): Mr. Stitt, when you roll a 6-inch Tee with its stem extending beyond the rolls, you get buckling effects. Can that be straightened?

Mr. Stitt: You can do your rolling with heat if you want to. The buckles can be taken out. The thing is you have excess metal there. And when you compress it by force buckles occur, just as in trying to straighten a deformed member. If you try to straighten a badly bent member with a bulldozer, you will get a lot of buckling. You can straighten it with heat, or if you already have buckles you can straighten those out with heat by knowing where and how to apply the heat.

I have a few slides that I would like Henry to show on this. Fig. 15 pictures two badly kinked members. They have an 8-inch width flange.

In Fig. 16 the section with buckled flanges is one that they tried to straighten with force. The other section unstraightened lies to the right.

Figure 17 shows the same two pieces, straight. The one at left has a 20-foot section, welded in to repair the damage done by force. There again, if the welder welded from one side right straight across to the other side, the chances are you would have shrinkage and distortion at that point. That, of course, can be straightened out, too.

Figure 18. Here are some more pretty nice bends with local buckling in members 80 feet long. Those sections have 8 inch flanges also.

Figure 19. Those same members are in this group of four, straight and ready for driving. The contractors driving the piles thought they would wiggle all around, and that brings up another point. In mechanically straightening a piece, you set up a phenomena known as the Bauschinger effect. Mr. Jackson mentioned it yesterday. What is the Bauschinger effect? Well, if you deform metal cold, and it takes a permanent set, it will be easier to straighten back than it will be to deform it further in the same direction.

I explained this to an engineer a few years ago, and he said that that explanation is something he never understood before. He said that all of us have driven a nail or a spike and had the whole thing bend, and when we pull it out we can straighten it nice and straight, but that when we try to drive it in again it frequently bends. That is because of the Bauschinger effect. The piece is unstable. It is a proven fact that 100 degrees Centigrade, the temperature of boiling water, for 24 hours, will wipe a good deal of that out. It restores the metal.

Five or six hundred degrees wipes it out entirely, and puts it back into the original condition. When we flame straighten these members, they are stronger than they were originally. The metal has been heat-treated. They are stronger than they were originally and no harm has been done to regular structural steel by applying the heat. Now, if you get into Carriloy T-1 that was mentioned the other day, we will have to study that one further.

Chairman Wood: Gentlemen, I hate to call an ad-
journment, but we are running over our time. This after-
noon at two-thirty, we will meet in the same place to have
an open discussion. I know there are quite a number of
questions that haven’t been asked yet. I don’t see why
we can’t start out the afternoon at two-thirty by con-
tining and finishing discussion on this part of the
program.
(The meeting recessed at twelve-thirty o’clock.)
Informal Discussion on Current Engineering Topics

The meeting reconvened at two-forty-five o'clock, Mr. T. R. Higgins, Director of Engineering, A.I.S.C., presiding.

CHAIRMAN HIGGINS: There was such an interest in this morning's panel discussion that I know there are questions left to be answered. I am going to suggest that we start with questions that would be directed to this morning's speakers before we introduce any other topics that might come up on the floor.

Has anyone a question they would like to direct toward this morning's speakers?

As you rise to ask your questions, please speak loudly so they can hear you in the back of the room.

MR. TOOF (Providence Steel & Iron Co.): My question is in regard to splitting beams. When you split them, so far as I can find out, you don't know which way they are going. Is there any way to avoid or minimize the work of straightening usually required?

MR. STITT: It will help a little bit to cut three or four feet, leave a tie of an inch or so, cut some more and leave a tie, repeating this for the full length of the beam. The short tie sections are then cut later. This doesn't help a great deal, but it will help some, because as each half cools from the flame-cutting, it is held by the other half of the beam. We sometimes do that on cutting plates to help hold them, but in general, I think cutting the beam and then performing the necessary straightening is required.

Beams with different geometries, made at the same mill, will react differently, and I am told that the same beam section from different mills will act differently. In other words, if you cut down through the middle of a beam web, sometimes the T-sections thus obtained will bow out away from the cut, and others will go just the opposite. Now, if the parts are going to bow out, some additional shrinkage along the edge of the cuts will pull the material back. This can be done, by putting two torches with pre-heat flames on the same carriage that is carrying the cutting torch. The pre-heat flames on the two torches serve to shrink the metal along the stems of the resulting T-sections. If the split beam tends to bow the other way it will be necessary to apply the heat out at the surface of the flanges opposite their junction with the web.

Now, that is the only solution I know that can be used. You will have to experiment to find out the direction the cut pieces are going to go, and then plan your operation to overcome the bow caused by locked up stress. These are locked in at mill and will bow the parts the same way. With a few trials you can find a place that you can apply additional heat as the flame-cutting is done to straighten them.

MR. GODDARD (Apex Steel Corp. Ltd.): We had a problem like that, Mr. Stitt, which we have settled pretty well by having to split quite a lot of beams. By supporting three torches on our carriage, a cutting torch at the center, and two right angle torches on the center line of the outside face of each flange we have no problem at all in keeping them straight.

MR. STITT: The main cut is in the web, and you use two torches with the flame, applied at the outside face of the flanges opposite their junction with the web?

MR. GODDARD: That's right.

MR. STITT: Heating those, adjusting the size of the flames, and carrying it along at the same speed of 18 inches a minute or whatever the carriage is moving would straighten the cut pieces that tend to bow toward the cut in the web. The additional heat on the outside holds them straight.

CHAIRMAN HIGGINS: I wonder, Ray, if you can draw any conclusions as to the magnitude of rolling stresses, residual rolling stresses, from your last statement? I would, off-hand, jump at a conclusion that since you put about the same amount of heat on the flange as you have along the cut, that there must have originally been very little rolling residual stress, since it takes about the same amount of heat application at the center and at the flanges to keep it straight.

MR. STITT: The last metal to cool—and that would be at the juncture of the web and the flange—will go into tension. Remember the slide I had this morning, with the heavy shaft, and a cylinder slipped over it, welded around each collar. The last metal to cool was the shaft, and the last metal to cool goes into tension whether in-
ternally in a beam or in the shaft. I have no measurements of the amount of stress that occurs in a rolled section. It would depend upon the cooling. I have seen references in literature that show there are very high stresses in rolled sections. I don’t lay awake nights worrying about them, nor do I think any of the rest of you do. If there are high stresses, they haven’t given us any amount of trouble and we don’t need to worry about them.

The actual edge of the cut, of course, is heated from the oxidation of the metal and there is a band on each side that probably extends half an inch or even an inch in width. It is heated up to 300 degrees along the cut because of the pre-heat flames and because of the actual cutting. It will be heated sufficiently to go into tension.

CHAIRMAN HIGGINS: Not to labor the point, but because it is a subject that has been receiving some discussion in the Lehigh Project Committee—as a result of splitting down the middle, without any flame on the flange, when the halves come apart, the edge is convexed. Is it not reasonable to assume that that is an indication of considerable locked up rolling stress in the flanges?

MR. STITT: It would be better to take a splitting saw and mill down through the web without any application of heat and see which way it goes. If it goes the same way, it would be because of the locked up stresses.

CHAIRMAN HIGGINS: But we would expect that the heat from the splitting would tend to shorten that cut edge rather than lengthen it.

MR. STITT: Yes, and it could well be the pulling medium.

CHAIRMAN HIGGINS: Another question?

MR. FOEHL: Mr. Stitt, on straightening material, you mentioned that you didn’t think heating to straighten it, or even working it back and forth with heat was going to be harmful due to locked up stresses, which I would go along with, but a lot of fabricators frequently use an air-water vapor gun in combination with heat to straighten material. In fact, in ship building the Navy has used it a lot in plate work, where they have a stiffener on four sides and the center part of the plate buckles. They heat it up and then use an air-water vapor gun to circle around that metal. I was wondering, what is your opinion of that? I would think that the quenching effect of the water would be detrimental and would perhaps leave the steel in a martensitic structure instead of pearlitic, making it brittle.

MR. STITT: The answer to that question is that I definitely feel it is better not to apply a water quench to the heated metal. After it has cooled down past 600 or 700 degrees, that will do no harm to cool it. It might get the action a little bit faster but not enough faster to require the application of water and, as A-7 steel does not have a definite carbon, we don’t know just what the condition will be when we apply water to a heated section of A-7 steel.

The ship plate that was used in the latter part of the war had more controlled chemistry, and they found no harm in applying a spray of air and water to the plates. On A-7 steel, I prefer not to use any additional cooling medium. If you apply the heat properly, it isn’t needed. It wasn’t used on any of the work that was shown in the slides this morning.

MR. WOOD: I have a couple of questions to ask Mr. Myers and Mr. Perry. I don’t know to whom to address these questions, so I will address them to both.

The first question is: What is the reason for using web splice plates on column splices?

MR. MYERS: The web splice plates are used for the double purpose of lining up the column and holding the web perfectly in line, one section above the other. They also serve as a part of the column splice to carry a portion of the shearing stress that comes from the wind loads on a column.

Anything that you want to add, Mr. Perry?

MR. PERRY: I think I know what Mr. Wood is driving at. Once you get your load on, there is little need for web splice plates. They are all right as Mr. Myers said for the erection process. I think we have got to have them. As a matter of fact, we only bolt them. We don’t weld around them in the field. I don’t think there is any disagreement there, is there, Mr. Myers?

MR. MYERS: I don’t disagree.

MR. WOOD: Well, it would appear to me—not trying to cause an argument here—that holes in the flange splice plates should align the column without any assistance from web splice plates.

My other question has to do with the city of Houston. I understand that the reason there is so much welding there—one of the major reasons—is because of the objection by the citizens of that city to noise. What I want to ask is, if they object to noise, how can they get by with using impact wrenches on the bolts? I think they are just about as noisy as rivet guns.

MR. MYERS: It happens that the noise from the impact wrenches in tightening up bolts is about one-half the noise of driving up rivets. They do stand for the use of impact wrenches.

I would like to observe also, in answer to the other question, that Mr. Wood forgets that the web of a column is not always in the center of a column. As a result, the flange splice plates will not necessarily line up the web.

MR. GILBERT (Ohio Structural Steel Co.): I have a question for either of the gentlemen who would care to answer. It refers to two common types of welded roof trusses. One type of design uses T-sections top and bottom, with verticals and diagonals of angles lapped on the stem of the Tee without gusset plates; the other
design uses rolled beam sections top and bottom, with the web members, also of beam sections butted against the flange of the top and bottom chords. In one case, you pull length-wise along the weld. In the other case, you pull at right angles to the weld. Which type is more preferable?

MR. STITT: I will make a comment. I won't answer the question. As far as shrinkage, you will have about the same type of shrinkage with both of them. I have found that a general rule is that you will have shrinkage of approximately one inch per hundred feet. You will have to allow for this shrinkage in order that your truss will end up the right length. It doesn't matter whether the truss is 75 feet long or 25 feet long. Where you run welds across the chord members, there is an overall shrinkage. You must allow for that, or your truss is going to be short.

I will pass on the main question to one of the designers over here.

MR. GILBERT: I have another question: What is the best arrangement of welds for a joint, placing the welds longitudinally, or does it make any difference?

CHAIRMAN HIGGINS: From a strength or appearance standpoint? We are talking about fillet welds. I am going to take the liberty of passing that along to Mr. La Motte Grover since he is Chairman of the Welding Research Council.

MR. GROVER: I think the answer to that question is to say that in the lighter truss, the first type of design mentioned, with T chords and lapped angles, is probably the most practical design. In the heavier truss—the type of truss with chords and web members all made up of rolled shapes—the second type of connection is more practical. Carl Kreidler is here from Lehigh Structural Steel. I know that they tested medium size trusses made up with rolled section chords in web members, and I know Austin Company has tested them. They certainly have withstood the tests that have been made on them. So, I think it is just a matter of economy and practicality, depending on the size of truss and the loads to be carried, that determines which is the best design.

MR. ARNTZEN (Mississippi Valley Structural Steel Co.): Mr. Perry's and Mr. Myers' talk this morning indicate a good deal of expensive research for the benefit of all of us. However, a good many of us don't operate in as favorable weather conditions as Houston. I don't know if these gentlemen can answer, but I would like to ask Mr. Dill of American Bridge, who has had a lot of experience in cold weather welding. Up our way and further north, we have to erect steel in all temperatures. If for any reason the rolling mills, or the architects, or someone should miss the schedule by a month and we are forced to erect, by welding, a building in the middle of winter, it might have to stand idle for weeks. We have had experience on coal bunkers where a good deal of welding was required. How do we erect when it gets down below zero?

MR. DILL: There are two answers to that question. One concerns the framed structure, columns and beams, where the welding is concentrated at the joints. The other concerns the plate-type structures such as the coal bunkers that Mr. Arntzen mentioned.

In our work, we have been guided by the requirements of the Institute's Specification and the Welding Society Building Code on welding at low temperatures. These require that, when the temperature drops below freezing, the steel in the vicinity of the start of the weld be brought to a temperature warm to the hand, and that all welding should cease if the temperature drops below zero. The first requirement is in recognition of metallurgical conditions. The second one is in recognition of the fact that a man can't perform welding skillfully when he is bundled against extremely cold weather. The matter of pre-heating is often taken care of by ordinary gas torches. On one job a carbon arc torch has been used to good advantage to avoid the necessity of bringing in an extra line of hose, torches, and gas cylinders to perform the pre-heating with oxy-gas torches. That scheme worked out well on building construction where the pre-heating was across a beam flange to column joint, or at the start of a beam web to column weld.

In plate work, such as coal bunkers, the problem is exaggerated by one other consideration, and that is that the temperature differentials and the corresponding changes in dimensions are exaggerated by the extreme cold, and we are faced with the problem of an extra 40 or 50 degrees of thermal expansion or contraction in the structure. We fasten the material together at a low temperature, and when it warms up, some parts may want to take a different shape than they have at a low temperature. The plate work is also more susceptible to the tearing type of fracture that we have seen in ship failures because of the size and shape of the parts.

The use of pre-heating on such work would have to be handled very carefully to avoid building up a large amount of expansion along the weld line which would not be matched by corresponding expansion but through the rest of the structure.

CHAIRMAN HIGGINS: Mr. Kreidler is in the audience. He also has to weld in cold temperatures.

MR. KREIDLER: We find that the outline as given by Mr. Dill is about the way we perform. In answer to the gentleman's question, you usually have a job in Boston in the winter time and in Florida in the summer time, and you are simply faced with doing the best you can with the weather. You have to tie up the job if you have too cold weather. But you usually have enough good days to follow your erection crew, and usually your welding does not tie the total job up.

CHAIRMAN HIGGINS: I wonder if I could ask Mr. Dill
and Mr. Kreidler if as a result of your experience you "ve,
three-quarters STETIN A: That is the in
"ve,or) of that 
Vt'eather filled
lnake
all in one pass. So, I say even though we might
be willing to weld, say, beams and rolled shapes,
32, and particularly as it gets
words, as you weld across the
extension bars before we answer?
the Code.
HIGGINS: Any other questions?
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tnink
a groove weld from the inside face of the
than
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with the bars.
HIGGINS: That is a question in two parts.
around zero, the ex~
weld
Yes, I would say that
is where he finishes his
dOVt'n beloVt'
54 •
three-fourths
to zero
it
about the merits of extension bars
ex~
start at one edge and weld toward
sequence or welding direction. Suppose we have a plate
with a double V-weld, as I showed in one of my slides
this morning. I feel that the weld should be started at
the edge and carried over to some point in the middle of
the joint. Then, the welder can jump his arc to the other
edge and weld back towards the middle. By doing that,
you can get good fusion at the start, over at the edges
of the plate. If you start at the middle and weld toward
the edges you are going to probably burn out a section
and make it difficult to have full cross-section. If you
start at one edge and continue welding right across in
one direction on a thick plate, chances are that the weld
will be filled up at one edge and only three-fourths filled
at the other. In other words, as you weld across the
puddle is harder to control. It will flatten down more
and you won't have an even weld all the way across.
My other point is this: By going from one edge to the
middle and then coming back in from the other edge,
one crater can be welded over the top of the other crater.
You can have a full cross-section of weld all the way
across and the straightness of the member will be
maintained.
If you are going to weld from one edge to the other,
or from the middle outward, yes, you should have some
kind of a bar to run your weld out on. But if you train
your welders to work from the outside in, the structure is
a lot better. If you follow this sequence, you will have
compression at the edges, and tension in the middle
where the tension will do no harm. So, I feel that ex-
tension bars aren't necessary.
MR. DILL: There is one thing that I would like to add
to Mr. Stetina's question. That is that these extension
blocks are not the only requirement of the specifications.
The specification states that on material three-quarters
of an inch or under, the edges of the weld or edges of
the plate at the end of the weld shall be chipped out and
carefully rewelded to eliminate the lack of fusion that is
likely to exist at the start and finish of the weld, so that
it is not a clear-cut case of using the extension blocks
or not.
The picture Mr. Perry showed does not come strictly under the control of the specification requirements that Mr. Stetina asked about. The specifications intended to take care of a situation that exists at the start and end of the weld. If we were to take a cross-section through the double-V weld discussed by Mr. Stitt, it is likely that each pass will tend to run out at the edge of the plate and as we lay additional passes each cascades out in the same way whether we start or whether we finish at the edge. Mr. Stitt’s suggestion of starting at the edge and finishing at the center is sound. His suggestion of reversing passes is very sound, but there is in the finished weld, even when the starts are made at the edge, a lack of soundness in the zone at the edge. The extension blocks, which amount to two blocks at each end of the weld, just tacked on allow this condition to be dissipated out beyond the edges, and then cut off and thrown away.

They are an expensive device. On a 10-inch wide plate, running a weld an inch into this block or that block adds 20 per cent of the weld metal that goes into the joint. Chipping this out and rewelding is also expensive, particularly when it is considered that the best job of rewelding would be accomplished by turning the piece up on edge. The requirement is in the specifications to provide welds that have 100 per cent soundness within the width and thickness of a butt welded member.

CHAIRMAN HIGGINS: Is it your feeling, Mr. Dill, then that that provision ought to be retained in the specifications?

MR. DILL: Yes, as long as we are going to allow the weld to carry the same stress that we permit in the base metal.

MR. STETINA: I would like to ask Mr. Dill why is there a line of distinction at three-quarters of an inch? If the practice below three-quarters of an inch is to chip out and weld and that is considered good workmanship, why not continue that practice above three-quarters?

MR. DILL: The three-quarter inch limit is an arbitrary point of division. It could be five-eighths, or it could be at one inch. As the thickness of the plate increases up to two, three, and even four inches, this cascading back becomes large and the amount of cutting out and rewelding would be major. It would actually be a job for cutting torches to trim out all of the potentially unsound metal. So the distinction in methods is largely one of economics. This method under proper control could be used on the thicker plates, but it requires very careful control of the cutting out or chipping operation to make sure that all potentially unsound metal is removed. The extension blocks make certain that it is removed.

MR. STETINA: From what you said, Mr. Dill, I would say that our problem of quality is related to the depth of the groove which, of course, determines the number of layers rather than the thickness of material.

CHAIRMAN HIGGINS: By that you mean that three-quarters of an inch would be too thick if it were going to be a single-V instead of a double-V. Is that what you mean?

MR. STETINA: In other words, a one-inch material with a double-V would not be comparable to a three-quarter inch with a single-V.

MR. DILL: That’s quite true, but if we are going to try to split the specification that fine, we will have an involved document.

CHAIRMAN HIGGINS: You have something to add there, Mr. Stitt?

MR. STITT: I am opposed to trimming out the metal, which can be deposited soundly by a welder, and putting more metal in there which is going to produce tension at the edge of the piece, where we don’t want it. I definitely feel that it is a lot easier to train a welding operator to put sound metal in at the start. Any cutting that is going to be done removes metal and you turn around and put metal back in. The last metal to cool is in tension. If you have most of your joint cool, your quarter or half inch of metal placed at the edges shrinks and puts tension there, which is exactly what we don’t want.

Now, there is a simpler way than putting the extension bars at the edges. The bars have to be beveled or prepared similar to the joints. Instead, on real thick pieces, you can take a small piece of quarter inch plate and bend it across the center, making a little angle shaped sloping trough, which can be put at the end. You can start your weld up in that sloping trough and weld down and then across to the middle of the butt weld and do the same on the other end and then trim them off, and you won’t have one inch of solid weld metal to be deposited out there just to get the weld started. You can flame cut and trim that off, and on an inch and one-half and heavier pieces, that is a desirable way to do it.

MR. KREIDLER: When you flame cut your extension bars, is there a possibility that your flame will notch the edge anyway and you will have to put a small butt weld on the edge of the piece to get a full section?

MR. STITT: That depends on the skill of the operator. He can get practically as smooth a cut as with a machine. The average operator will not give you a real smooth cut on the side. You will have a notch effect.

CHAIRMAN HIGGINS: Any further comments on this particular point before we take another question?

DR. WEBB: I take it from Mr. Stitt’s remarks, then, that he favors the extension?

MR. STITT: The extension with a good flame cut edge along there is a fine way to do it. It can be deposited soundly without needing the extension bars.

CHAIRMAN HIGGINS: Are there any further questions on this subject of welding? If there are none, I will entertain questions on any other subject that is of interest to anyone in the audience. This is just an entirely in-
formal discussion period for whatever problems interest
you on which you would like to hear from others in the
audience who may have had similar experiences.

Mr. Hall: I would like to know whether we have any
further information on the probable cause of the failure
of the welded bridge up in Canada?

Chairman Higgins: The Duplessis Bridge? That
came in for a lot of discussion last year. Is George Lamb
here? I think George it might be well, in case there are
different people in the audience this year than last, that
you review just what you know occurred.

A REVIEW AND ADDITIONAL FACTS IN REGARD
TO THE FAILURE OF THE DUPLESSIS BRIDGE
AT THREE RIVERS, CANADA
GEORGE W. LAMB

Last year at the meeting in Pittsburgh I discussed the
probable cause of the failure of the Three Rivers Bridge
in Canada. This was reported in the AISC National
Engineering Conference Proceedings of 1951. It was
suggested at that meeting that the failure may have been
caused by an expanding concrete floor that pulled the
girders in two, and that the concrete in the floor should
be tested for non-thermal expansion.

The engineers working on the cause of this failure,
and the Institute engineers, had enough confidence in
this possibility to obtain a specimen of the concrete from
the Three Rivers Bridge floor for tests. Tests on the
expansive properties of this concrete are also being con-
ducted in some of the Universities of Canada and in other
places. However, the results of these latter tests are un-
known to me. The tests that I have knowledge of, show
that this concrete had very serious non-thermal expansive
characteristics and verifies the postulate made at the Pitts-
burgh meeting.

There were two bridges at this location and both are
described in the 1951 Proceedings. Twenty-seven months
after construction of these bridges, the girders in both
bridges cracked at temperatures considerably below zero,
but complete failure did not occur. Approximately twelve
months later when the temperature was again well below
zero, the West bridge failed and four spans of the structure
fell into the river. The initial break occurred about
as shown at line "C", Sketch 4, 1951 Proceedings. There
was no heavy traffic on the structure at 3:00 A.M., the
time of the failure.

As is usual in such cases, the cause of the failure of
any structure is difficult to determine and prove. This
is especially true when the suggested cause of failure is
different and contrary to design assumptions and con-
struction practice that has been accepted for many years.
In this particular bridge such things as the following are
beyond most of our personal experiences: 1. The stress
creep in the girders under very slow application of tensile
loads. 2. The probable notch effect caused by sharp peaked
dead load stresses over the piers. 3. The retardation of
increase in length of expansive concrete caused by some
means of restraint. 4. The effect of the anti-creep lugs
on the top of the girders. 5. The actual moment and
diagonal tension stresses in the girder near the piers.
6. The probable redistribution of dead load stresses when
parts of the girder are stressed beyond the elastic limit
by large tensile stresses. 7. The effect on the ductility of
the steel in the webs caused by stiffeners. 8. Determina-
tion of the past "expansive" history of a material as in-
consistent as concrete from laboratory tests on samples
four years old. 9. The effect of different elastic limits in
the flange and web steel on the web stresses. 10. Last but
not least and most difficult of all—very few people be-
lieve that concrete can expand non-thermally or that this
expansion has a great destructive potential.

Non-thermal expansion of concrete is not spectacular.
It is slow and rather steady, and its effects are very dif-
ficult to recognize. When a steel bridge has an expansive
type concrete floor, the results are usually observed in
movement of the slab on the beams, expansion joint
failures, torn diaphragms, torn webs of end floor beams,
and displacement of bearings. Some floors ride over the
splice plates and rivet heads, leaving a gap between the
floor and the beam. Concrete spans show the effect, by
split pier caps, spalled joints, loose or squeezed expansion
joints, increases in camber, displacement on pier caps,
diagonal tension cracks, and disintegration of the
concrete.

In large concrete dams, non-thermal expansion results
in the destruction of generator and machinery founda-
tions, failure of overflow gates, and upstream movement
of the crown of arch dams. This is perhaps the cause of
much of the foundation grouting that has to be done as
maintenance.

On the other hand, freezing expansion is reported to
be sudden, violent and rather spectacular. Freezing is
perhaps responsible for much of the scaling of concrete
slabs, and the very rapid disintegration of some concrete
in cold climates.

The natural inclination of a person when faced with
the above defects is to endeavor to correct them by
strengthening the structural parts. Such measures as
adding bars in pier caps, redesigning the connection of
expansion joints to the supporting structure, and anchor-
ing the floor to the supporting steel, have been tried.

In this bridge, lugs were welded to the tops of the
girders to prevent creep of the floor on the girders. Very
heavy expansion joints were welded direct to the girders
and connected to the first floor beam away from the
joint by several beams run parallel to the center line of
the bridge. These connections were capable of trans-
mitting, to the girders, any loads applied to them from the floor.

From the appearance of the fracture, the initial crack in the steel girder started in the top of the web, therefore all of the following stresses were calculated at the edges of the web. The dead load moments are as shown on Figure 1. These moments were calculated using a 4.4 kip per foot load per girder. (Note the moment is largest near the points of fracture). Tests made to determine the actual stress over the intermediate supports of continuous girders indicate that the stresses do not peak as much as the theoretical moment curve would indicate. The stress over the supports follows more nearly a transition curve somewhat similar to that shown on Figure 2.

Figure 2 shows the stresses caused by dead load moments at the edge of the web. In addition to the moment stresses, the webs must also accommodate shear. The total tension in the top of the web, resulting from moment and from shear, or diagonal tension, is shown at the support as a line above the stress caused by moment alone. These stresses, produced by dead load, are well within the safe working stress of any type of steel and it is impossible that they alone could have caused the failure. Therefore, stresses produced by an expanding concrete floor were added to the dead load stress and are shown at the upper left corner of the figure.

Calculations were made to determine the effect of a direct tension load on the girders, when combined with the dead load stress. The tensile load was applied at the center of gravity of the girder cross-section. This assumption is somewhat in error, as the longitudinal load was applied at the expansion joint and lugs along the top flange of the girder.

Using as a basis the variation in cross-sectional area of the girder flanges and webs, it was found that a load of 325.1 kips would elongate one girder 2 inches in its total length. As there were approximately 5,000 sq. in. of concrete in the cross-sectional area of the concrete floor acting on both girders, there would be 2,500 sq. in. of concrete area per girder. Therefore, concrete compressive stresses of 130 psi would be sufficient to cause the girders to elongate 2 inches. The P/A stress in the steel girders caused by this load, which produces 2 inches of elongation, varies from 2,730 psi at the interior supports to 4,260 psi near the inflection points.

The P/A stress that would be produced if the girder acted elastically and was elongated 12 inches, or the equivalent of .00073 units per unit elongation in the concrete, was added to the dead load stresses shown on Figure 2. The total stresses are shown in the upper left part of the sketch. They are above the elastic limit of this steel.

If one point of this girder reaches its elastic limit, while the remaining parts are below it, all of the additional movement of the floor will be concentrated at that point. To illustrate: a test specimen of A-7 steel with a uniform cross-section 8 inches long will elongate under tension more than 22% in its length. If, however, this steel has a varying cross-sectional area along its length, the least section will reach its elastic limit before the remaining parts, and the specimen will break at its smallest or highest stressed section, at an elongation much less than 22% of the total length. It is very important that this fact be kept in mind when thinking of this bridge girder, which varies in cross-section and carries large bending and diagonal tension stresses.

From the curve, it can be seen that if a tensile load of six times (12 inches of elongation) the 325.1 kips is added to the dead load stresses, some of the steel is near or above its elastic limit, and a considerable part of it is above its creep stress. Six times the 325.1 kips, or 1950.6 kips, will elongate this girder more than 12 inches and will induce a stress in the concrete floor of six times 130 psi or 780 psi, which is well below its crushing strength. A tensile force of this magnitude would produce fracture in the girder at an elongation of about 12 inches.

Before the results of the tests on the concrete floor were available, but after the suggestion was made that an expanding floor may have been the cause of the failure, a commission was appointed to determine the cause of the failure. The results of their findings appeared in many papers and were reported in Engineering News-Record. They determined that the cause of failure was:

1. An unknown scientific phenomenon, or
2. Sabotage.

There was no mention that any consideration was given to the cracks that occurred the year before.

On September 12, 1951, and after the Commission's study and findings were made, a piece of concrete 20"x26"x11" from the Three Rivers Bridge floor was received for tests. It was cut into suitable test specimens, 3"x4"x15", and various tests were immediately started. The concrete was very dense with a weight of 151 lbs per cu. ft. It had a non-destructive or dynamic E of about 6,000,000 psi. The concrete was four years old when the tests were started.

Three of these specimens were subjected to the standard highway department wetting and drying tests for expansive concrete. The specimens in these tests are never completely dry, or completely wet or saturated. The 24-hour cycle consists of placing the specimen in water at 70 degrees for 16 hours and then drying in an oven at 125 degrees for 8 hours. This is supposed to be an accelerated weathering test. Under this treatment the specimens had an expansion of .035% at the end of 45 days and definitely indicated an expansive concrete. At the end of seven months the expansion had increased to .073% and was still increasing. All three of the specimens expanded the same amount. These tests will be discontinued at the end of one year, or when the concrete
has expanded 0.1%. (Note: .073% or .00073 units per unit expansion in just seven months of tests is sufficient to elongate the girders 12 inches, an amount sufficient to fracture them.) The tests reported were made by a Highway Department laboratory.

Practically all tests on expansive concrete have the objective of determining its durability. There are few tests, if any, that have been made to determine the forces required to prevent non-thermal expansion. Certain facts indicate that the forces necessary to prevent this type of expansion are above the crushing strength of concrete. The forces produced by the expansion are very large as evidenced by increased heights and upstream movements of the crowns of some large concrete dams made of this type concrete. Where expansive concrete has been used in bridge floors, and the expansion has been restrained by reinforcing bars, sliding friction, and expansion joints, the rate of expansion has been almost exactly the same for the restrained floor as for the laboratory specimen.

The effect of concrete creep, elasticity (if any), and other factors, resulting from restraint of expansion, cause little if any reduction in the amount of expansion of the concrete. Most experts on the action of this concrete believe that it will produce sufficient forces, if confined in one direction to cause failure by crushing. Most of them also think of this action as one of volume change, somewhat similar to that which occurs in the freezing of water. In actual use, expansive type concrete is usually restrained, but the expansion is not prevented by this restraint. No suitable explanation has been found for this unreasonable behavior of expansive concrete.

Assuming that expansion in the Three Rivers Bridge concrete specimens was accelerated by the tests, to four times its normal rate, considerably more than can be expected, a curve such as shown in Figure 3 can be drawn. The characteristics of this curve are typical of some concretes. Concrete is not a consistent material and its properties of course vary over wide limits. Some of it expands at a much more rapid rate than shown, and some expands very little, if any, when subjected to the wetting and drying tests. As an example of this material's inconsistency, one specimen of this concrete was stored at 74° with an RH of about 60% and had a shrink of .025%. In all known cases expansive type concrete expands more rapidly in the first year or two than it does later.

The curve shown in Figure 3 is on the conservative side, as no consideration was given to the probable rapid expansion during the first years. The slope of the irregular line was determined from laboratory tests on the concrete when it was four years old. However, the curve is sufficient to demonstrate the point. The concrete is now about 1,800 days old, and it is still expanding. The laboratory test at an assumed four times accelerated rate over a period of 210 days produced an expansion of .00073 units per unit. From this data the slope of the expansion line was determined. It would, when extended into the past, reflect the past expansive life of the concrete. When the girders first cracked in 1950 the concrete was about 800 days old, and without doubt had an expansion at least as great as shown on the curve, regardless of its degree of restraint. At the time of the complete failure in 1951, the expansion was at least as great as shown on the curve. The break that caused complete failure occurred when the concrete was 39 months old and the total expansion amounted to a possible .0009 units per unit.

The first cracks that occurred and the subsequent repair of all web and flange splices, relieved the girders of some stress especially near the complete break. The amount of wetting and drying expansion, or non-thermal expansion, would have been sufficient to cause failure in the steel girders. However, the failure may have been delayed a few months due to creep strain in the steel.

The fact that failure took place at low temperatures leads to the belief that freezing had some effect. No freezing and thawing tests that I know of have been made on this concrete to date. The Bureau of Standards made expansion tests on concrete exposed to freezing and thawing, and the results are published under their paper RP 2000 and republished by the Journal of the American Concrete Institute, February, 1950. The results of some of these tests are shown in Figures 4 and 5.

Figure 4 shows a single cycle of freezing and thawing of a 100% saturated concrete specimen. The increase in length shown here, if it had occurred in this bridge floor would have produced an elongation of 38 inches. There is no evidence that this amount of expansion did take place, but some part of it in all probability occurred.

It is doubtful if all of this bridge floor could have been 100% saturated. Some parts of it could have been, as it was exposed to freezing and thawing cycles and this does produce 100% saturation. In studies of the weathering of some rocks, it was found, many years ago, that when the rock was exposed on all sides and subjected to repeated cycles of freezing and thawing, water was forced into the inner pore structure with 100% saturation. As a final result disintegration of the specimen occurred on further freezing.

Concrete specimens that are 85% saturated act as shown in Figure 5, when exposed to repeated cycles of freezing and thawing. Among the experts, this action is known as "ratchet action." Concrete having measured 85% saturation is not uncommon in structures exposed to the weather. If cycle eleven in this sketch had occurred on this bridge floor, the floor would have increased in length by 2½ inches, and the increase would have been sudden.

Every fact known about this structure, proves beyond reasonable doubt that the non-thermal expansion of the concrete combined with some freezing of the concrete
ASSUMED ELASTIC LIMIT 35 K.S.I.

Remaining Stresses Similar

Combined shear and moment stress + P/A caused by concrete expansion of 0.00073 units/unit.

Cracks and break occurred about here

Top Flange

Bottom Flange

Direct Tension P/A

Combined shear and moment stress

Moment f_s

Top Flange

Bottom Flange

SPAN 1   SPAN 2   SPAN 3   1/2 OF SPAN 4

UNIT Stresses AT EDGE OF WEB

FIGURE 2

LAMB
FIGURE 3

FIGURE 4

FIGURE 5
floor caused the failure of this bridge. Everyone of us has had experience with the effects of expansive concrete, although we do not always recognize it. Its effects are indicated by failures of expansion joints, torn diaphragms, etc. We seldom, if ever, place the cause of these partial failures on the concrete where it belongs. These failures are usually placed on the steel, its design, its construction, its quality and its dependability. We now have positive facts proving that the failure of the Three Rivers Bridge was the result of an expansive concrete floor and that the steel girders, their design, their welded construction, and their quality of workmanship were adequate to accommodate all of the usually considered loads that could possibly occur on the structure. We should use these facts to protect ourselves, our material, and not least, the public and the engineering profession.

During the last twelve or fifteen years, some recognition of the destructive effects of expansive concrete has occurred. The almost complete failure of Parker Dam was proven to be the result of expansive concrete. The crown movement and increase in height of Cooper Basin Dam was the result of expansive type concrete. Reports on the destructive effects of this concrete have been published in many technical papers, in the Transactions of ASCE and Proceedings of ACI. The authors of these articles are in general engineers, and in general they are favorable to the use of concrete. Still, they, in the interest of their profession and the safety of their structures, have published hundreds of thousands of words relative to the non-thermal expansion of concrete. The expansive characteristics of some concretes are not a figment of my imagination, as it is well known and rather highly publicized. Millions of dollars have been spent by the people interested in concrete construction, in a vain effort to correct or control it.

We should see to it, that the steel structures in which we are interested are so designed and constructed that, regardless of the action of the adjacent concrete, complete failure cannot occur. It required many years before design and construction specifications took account of the effect of shrink and stress creep of concrete although these rarely cause complete failure. On the other hand, there is no design specification at present that considers the effect of the much more dangerous action of expansive concrete, although some material specifications make an effort to limit its expansive properties. This limit of expansion established by such specifications is in itself a sizable amount, being 1 inch per 100 ft., or .00083 units per unit. Reinforcing used in this type of concrete, if it acted elastically, would be stressed 30,000 psi in addition to its regular stresses. Structural steel firmly connected to this specification concrete will have additional stresses of 30,000 psi, if the ratio of concrete area to steel area is above 10.

CHAIRMAN HIGGINS: Are there any other topics? If not, before we adjourn, I would like to take this opportunity to express the thanks of the Institute to each of our speakers for the very fine help we had from them on this program. They certainly made the program mighty interesting for us and very instructive. Hearing no further question or topics for discussion — the meeting stands adjourned.

(The meeting adjourned at four-ten o'clock.)
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