WELDED HIGHWAY BRIDGES

James G. Clark
Balke and Clark, Consulting Engineers

Mr. Clark, fellow engineers and guests, it is an honor to be invited to a conference of this kind to speak on the subject of welded highway bridges. My experience with structural conferences has been in the main to plan them and arrange for oustanding men to deliver the lectures. My good friends at Illinois have always been favored by my absence from the program as a speaker. My many students have not been so fortunate. During 18 years of teaching a man is bound to pick up habits and possibly abilities, sometimes without realizing the development. My hypnotic power over students did not seem so important until I found that my lectures were putting me to sleep also. The use of slides has added benefit in this respect so accordingly I have brought along some slides to show.

It is impossible to discuss welded highway bridges in any broad sense in a matter of an hour or less, by any engineer, let alone a former teacher. Even limiting this to the field where welded bridges differ from riveted bridges, the subject remains too big. In order to accomplish the most it seemed desirable to bring out a few specific points that in the least would provoke some discussion out of which a few good ideas may come. To this end I have selected some slides that show pictures of, and drawings for, welded bridges, some of which have been in use for a few years, some in the process of construction or just finished, and some of which are proposed. A number of the latter were taken from papers presented in one of the three welded bridge programs sponsored by the James F. Lincoln Arc Welding Foundation.

To state the often repeated remark, a welded bridge should be designed from the very beginning with all of the possibilities of welded fabrication in mind. The more important characteristics or advantages of welded construction include the following. Connections capable of carrying large bending moments are normal and are a natural result rather than something to be considered as a special case; however, alertness toward possible stress concentrations should be ever present. More plates and fewer rolled sections are a direct result of welded fabrication. In general, effective areas can be located at a more beneficial position. Also right angle connections and bridge cross sections of rectangular shapes are not necessarily more economical than bridge cross sections of triangular or trapezoidal shapes.

Over a period of many years expressions for least weight depths of girders and trusses have been developed for riveted structures. Sometimes these have been called economical depths, although the economical depth and least weight depth may not be the same. Now it will be necessary for the structural engineers to determine what depths and widths should be used for the various spans and loadings when the bridges are to be welded. These depths and widths are not the same as for riveted structures. To take one example, a rather common expression for the least weight depth of a plate girder having a web so thin that stiffeners are used is 5.5 times the cube root of the bending moment divided by the allowable stress. For a welded plate girder having intermediate stiffeners the expression is the same except that the factor 5.5 becomes 5.0. As soon as possible sufficient studies should be made to enable the bridge designers to have at their disposal as many reliable relationships such as depth to span and width to span for welded bridges as we now have for riveted bridges.

Welded structures have advantages over riveted structures in the possibilities of saving weight, saving maintenance and painting cost, and giving a more attractive appearance; however, greater attention must be given to the steel which is used. The steel may or may not cost more but its chemistry and mechanical properties are of greater importance in the welded structure. This topic was covered very adequately yesterday afternoon and I shall say no more about it.

In designing and building welded bridges the engineers here have been more reluctant than those in Europe. Consequently we find that only very recently have we begun to have welded bridges in any significant numbers. At the present time the trend toward welded bridges is increasing rapidly and the slides that I shall show include just a very few. Let us start with the first one now and note some of the design features as we go along.

(1) This first slide does not show details but does indicate the achievement of horizontal lines which are considered so attractive now-a-days. This is the Market Street Road Grade Separation at Harris County, Texas. The roadway is 26 feet wide, the span lengths are 70 ft., 90 ft. and 70 ft. The longitudinal members are continuous over the center piers. It was open to traffic late in 1950.

(2) Here is another continuous type structure in Harris County, Texas, with spans of 54 ft., 72 ft. and 54 ft., with two twin structures each having a 28 foot roadway. It was also opened to traffic in 1950.

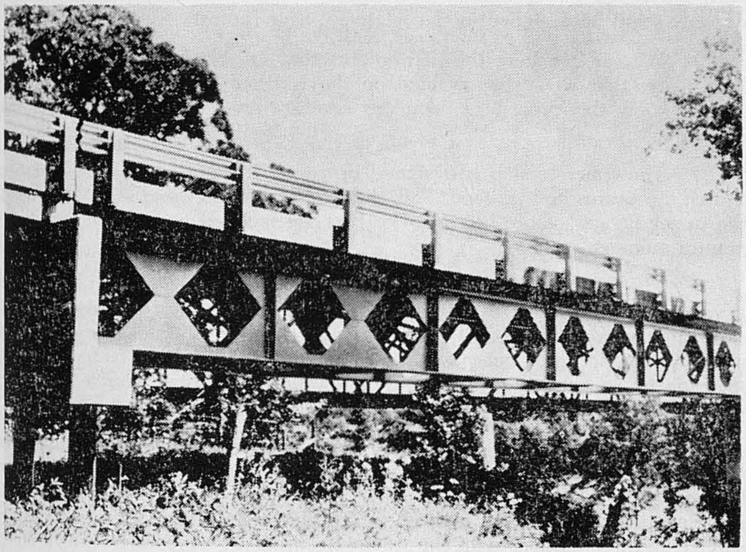

FIGURE 1

(3) (See Figure 1). The Texas Highway Department designed this 100 foot simple span girder. This is known as an expanded beam. The increased depth is accomplished by welding together a 36 WF 160 that has been cut to give a serrated web. This process has been used in England and many other parts if Europe rather frequently. It is possible to save considerable weight if the shear is low. After welding, the original 36 inch depth may be increased to as much as 60 inches. More commonly the shear limits the depth to 54 inches.

As you can see stiffeners are located two web holes apart. In some respects a structure of this type is very similar to a Vierendeel truss is one type of structure that can be designed in welded construction much cheaper than in riveted construction. Again for maximum economy the shear to moment ratio should be low.

(4) The next few slides are of the all welded Freeway Viaduct in San Francisco that will connect the Bay Shore Freeway south of the city to the San Francisco-Oakland Bay Bridge. The total cost of the entire structure will be in the neighborhood of $15,000,000 and will be the largest all welded highway structure known. It will require approximately 61,000,000 pounds of structural steel. The volume of traffic is estimated to be 90,000 to 100,000 cars per day. This column is one of a double-column bent. The main web is 36 inches, the flange plates are 24 inches. The outside of the flange plates are stiffened with 6 inch side plates and the web is stiffened at the middle with 6 inch tees.

(5) Here is the steel for one span and the two - two column bents supporting it. One column, two column and three column bents are used in different places along this viaduct, all of cantilever or rigid frame type of construction– having no bracing at all.

(6) Shown here is the steel grillage detail of a typical column. For years before welded bridges were designed, welding was used extensively in various shoes and grillages of footings. I am convinced that the voluminous welded repairs for railroad bridges is a similar forerunning indicator of many welded railroad bridges that will be designed in the relatively near future.

(7) We can see here the diaphragms that are being field welded to connection plates that were shop welded on the webs of the longitudinal members. As we all know this procedure decreases erection costs and permits larger tolerances in lengths|

(8) The concrete slab is designed in such a manner as to form an integral unit with the beams and girders. This slide clearly shows these shear lugs welded to the upper flanges of the beams. The lugs are inclined toward the direction of maximum moment.

(9) The longitudinal beams are being welded to the cap girders but the field welds have been kept to a minimum, as is indicated by comparison with the painted shop welds. Almost all of the welding on this job could be done in the shop by automatic and semi-automatic processes. Many welds 70-feet in length were made by an automatic process using a welding machine mounted on a self-propelled travel carriage that was placed on a 70-foot long box-beam.

(10) (See Figure 2). This is the overpass at Eleventh and Bryant Streets and shows long stretches of the steel viaduct ready for the slab forms to be placed. Before going on to another bridge I would like to mention a few items involving welding procedures that were used on this viaduct. The longitudinal girders were fabricated in three sections with the splices occuring wherever there was a change in flange thickness. The flanges were tack welded to the web and then welded by the full automatic or semi-automatic machines with a 3/8-inch fillet weld. Stiffeners were welded to the web with a machine and hand welded to the upper flange only. In welding flanges to the web both sides of the web were welded simultaneously to minimize distortion, starting at the center of the section and welding toward either end.

(11) Although this is not a bridge in this country it represents a type of design that is becoming more common. This and the following two slides are of the Pont de St. Cloud in France. The five center spans are each 31½ meters in

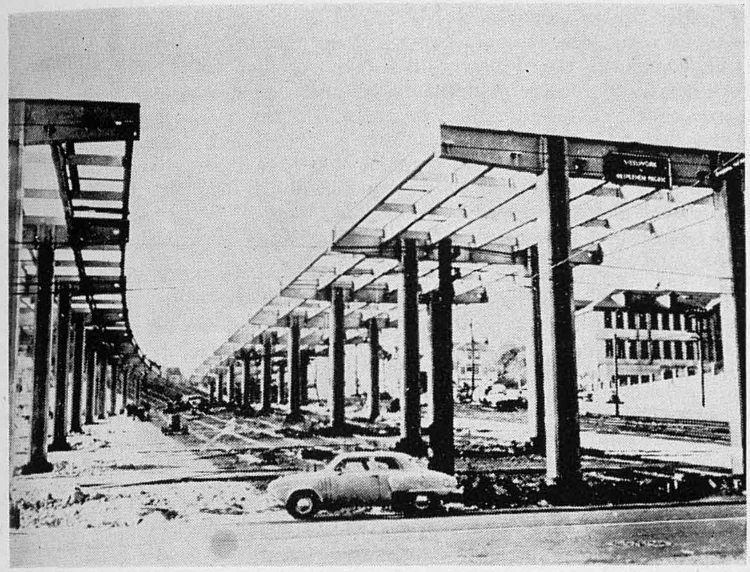

FIGURE 2

length. Particular attention was given to this fascia for reasons of appearance. It has nothing to do with welding but all bridge designers must consider attractiveness to whatever extent the money available will permit.

(12) This underneath view shows the slab being supported by longitudinal members only. Diaphragms are alternated with plate stiffeners on these longitudinal members. The number of pieces has been held to a minimum.

(13) Here we have a much better view of the diaphragms and can see that they are of a Vierendeel type or rigid frame type with the absence of any diagonal bracing either in the vertical or the horizontal planes. Also we can see the partial depth stiffeners spaced between diaphragms and vertical stiffeners. These are nothing more than short tapered plates.

(14) Possibly most if not all of you have received a pamphlet published by the Lincoln Electric Company of Cleveland outlining this bridge in considerable detail. It is the Benton Street Bridge over the Iowa River in Iowa City. The symmetrical span layout consists of five spans having lengths of 78 ft., 100 ft., 120 ft., 100 ft. and 78 ft. The structure is designed continuous for the 480 ft., end to end. The girders are 20 ft. 3 in. apart and serve as stringers also, thus reducing the number of stringers to two intermediate lines and collectively support the roadway of 24-ft. curb to curb. There is a sidewalk on the far side supported by brackets. The intermediate stiffeners are 5 in. ST sections with the flange of the tee outstanding from the web of the girder. These stiffeners are placed on the inside of the web only.

(15) The next eight slides are of the Rio Blanco Bridge currently being built in Mexico. My special interest in this bridge started when the design of it was presented by Professor Kavanagh, then of Pennsylvania State College, in the

1950 welded bridge program. At that time I did not realize that it would become a reality so soon. The program paper was not a detailed design and Mr. Camilo Piccone completed the design work in cooperation with Professor Kavanagh. Only a few changes were made in the basic design. As originally presented the span was exactly 250 ft. for the through arch bridge. This is a two-hinged arch with ties that act also as stiffening girders. By inclining the arch planes so that they join in the middle, 80 to 90 per cent of the top bracing has been eliminated. The plan view shows the grid floorbeams that directly support the slab and eliminate the need of stringers and lower diagonal bracing; furthermore, this floor grid acts to some extent with tie girder tending to reduce the axial in the latter under eccentric lane loadings.

(16) Of necessity the tie girder is inclined at the same angle as the box shape arch rib. An improvement on the original design can be seen in comparing this slide and the next one in regard to the rib section in the region where the two ribs join. Here you see the top plate is horizontal.

(17) Here you can see the top plate having a V-section that reduced the complexity of fabrication considerably. Another change is the matter of hangers. The rope or bridge strand hangers were replaced with two channels laced together.

(18) This view shows some of the tie girder already placed with an additional section being brought in for positioning. It emphasizes the inclination of the tie or stiffening girder.

(19) This close-up furnishes some unusual details. Not only does the web have stiffeners but the stiffeners have stiffeners or small triangular brackets. The web is 72 x ½ inches, the stiffeners are 6 x 7/16 inches. Note that the flange plates lit in a horizontal plane.

FIGURE 3

(20) (See Figure 3). Here is the inside of the tie girder, it shows the vertical stiffeners and the horizontal plate located at slab height. Slab receives edge support from this longitudinal plate.

(21) This field storage yard has many of the grid floorbeams stacked ready for use. You can see the manner in which they are notched ready for mating. Each of these beams is intersected by three other beams and consequently has three notches.

(22) This floorbeam will have to be turned over before it can be mated with the ones that arealready in place. These are 21 WF 62 sections.

(23) This is the cross section for 120-foot deck span that is supported by the two box girders shown. It was submitted by Mr. H. M. Hadley and Mr. H. H. Johnson in the 1949 bridge program. Actually a structure very similar to this was built in King County, Washington. It is located 30 miles from Seattle. It is written up in the *Engineering News-Record* April 12, 1951 by Mr. Hadley. The thin steel plates were welded into the shape of a box girder and form the support for the 110-foot span built in Washington. The web plates are 66-inches deep and 1/4-inch thick. They are stiffened by diaphragms as shown in the cross section and longitudinal bar trusses as shown in the longitudinal sections. Shear connectors made of 2 x 1/4 flat bars were welded in pairs to the top plate. They are attached to a 30-degree angle and spaced 3 ft. 9 in. apart.

(24) Now I have two pictures and two drawings of bridges built by the Connecticut State Highway Department under the direction of Mr. John F. Willis, Engineer of Bridges and Structures, that represent what one might call more conventional design than the last two slides have depicted. Whether it means anything or not these structures have been in use for twelve to thirteen years. This bridge is part of the Hartford By-Pass in the town of Wethersfield over U. S. Route No. 5. It shows twin rigid frame of welded steel. The next slide will show the details.

(25) In the upper right-hand corner we can see a diagonal stiffener that is necessary there, but we note at the center column shown in the upper left-hand corner there is no diagonal stiffener because none is necessary. Such stiffeners can be easily fabricated only if the structure is welded. As this drawing indicates stiffeners are welded only to the compression flange. Along the top flange of the rigid frame in regions of tension the stiffener is welded to a 2″ x 1/2″ x 5″ bar that bears against the tension flange. This stiffener detail is shown in the lower right-hand corner.

(26) More of the Hartford By-Pass, but in the town of East Hartford. This single span rigid frame crosses Main Street. The next slide shows the details of this bridge.

(27) Because of the skew the web of the diaphragms are bent, then welded to the stiffeners which are normal to the web of the rigid frame. The shoe detail shown at the left has been designed with comparative simplicity.

(28) Here is a structure as proposed for a 120-foot span. It shows some weight saving possibilities for longer spans but I question the value for just 120-feet. The deck consists of asphaltic concrete applied to a 5/8-inch floor plate that is supported by stringers. The top flange of which has the shape of a Vee and thus can provide a double support for the floor plate. A stringer of this type can be fabricated by welding a rolled angle to a rolled tee, thus forming what might be called a Y-beam. If a steel floor plate is used, this plate becomes effective area for the stringer. In like manner the transverse floor-beams need to

be only Tee sections since the floor plate again provides the top flange area. Although a horizontal stiffener is shown in this drawing I wonder about the desirability of its location. There is no question but what horizontal stiffeners can replace the need of vertical stiffeners—sometimes to advantage. In this case, it might have been better if the horizontal stiffener had been located at the elevation of the bottom flange of the stringer. Here it would provide temporary support for the stringer and, if attached to the bottom flange of the stringer, it would have additional stiffness thereby and be of greater value in stiffening a thin floor-beam web. The best specifications concerning horizontal stiffeners that I know of is contained in the specifications for crane girders. Almost always these light weight box girders that support traveling cranes are designed with horizontal stiffeners, frequently in combination with vertical stiffeners. Of necessity they must be light weight and invariably they are of welded construction.

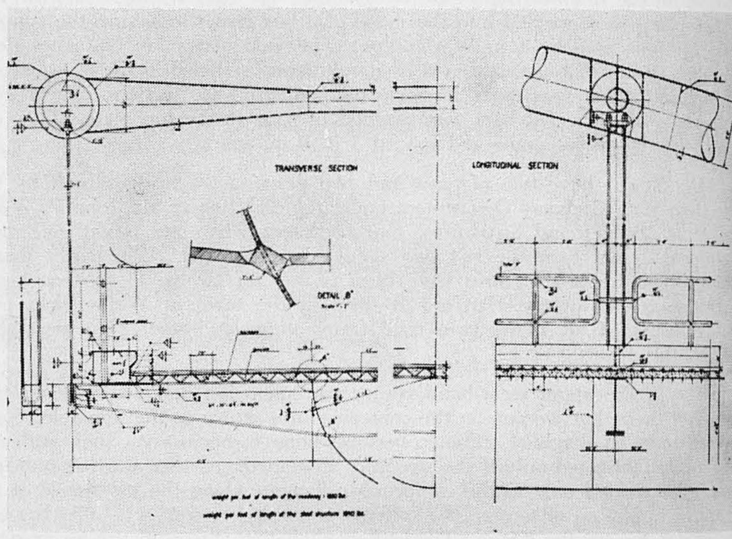

FIGURE 4

(29) (See Figure 4). Especially for compression members the pipe section has maximum strength for a given area. We have seen from time to time many proposed methods for the utilization of pipes in various structures. Here we see a proposed 120 foot truss designed with pipes. The unusual item is the suggested pipe adaptor shown in the lower part of this drawing. It is true that such a bar could easily be rolled and cut to any desired length. If such bars were available the complexity of gusset plate attachments would be vastly reduced. A difficulty of splicing pipes is that of getting proper backing material or providing any weld except that on the outside. Although in general butt welds are preferred, this pipe adaptor bar would enable one to design a very simple pipe splice with only outside fillet welds.

(30) Again we see a truss for a 120-foot span, but this one has a triangular shape utilizing two top chords and a single bottom chord. The shapes suggested here are not now available; however, such shapes are not necessary for a triangular

truss. It is true that if they were the fabrication process would be simplified. Nevertheless, a triangular truss of this type, properly designed, will result in a considerable saving in weight. The need for bottom lateral bracing is an absolute minimum. The resulting structure has exceedingly high torsional strength, consequently the transverse distance between top chords may be reduced. It is obvious that no transverse cross bracing is needed in any vertical plane.

(31) This shows more detail of what a triangular truss consists. Being different this proposed structure has three triangular trusses as seen in the cross section to the upper right. The top chords of the center truss serve also as the inner top chords of the outside trusses. Having three such trusses the need for floorbeams and cross bracing does not exist. The only section suggested here that is not commonly stocked is a bent plate, or if one prefers—a 60-degree angle. The amount of edge preparation would be reduced if 60-degre angles were available.

(32) You must remember that my experience includes research and teaching, as well as practical design. Being an individual with this kind of a background I cannot help but let my thoughts wonder into the realm of the future when I am speaking to any group. This situation changes when I am in the process of designing or detailing a structure—my common sense and the dollar sign of costs hold sway and submerge the tendencies to speculate. With this in mind I hope you will understand the reasons for bringing the next two slides. Possibly you may wish to throw some of the others in the same category, I leave that to you. Although this cellular structure was suggested for a 120-foot span, the designer brought out the fact that this type of construction would be of greater value in longer spans. Thin walled cells, 4″ x 4″, placed side by side, form a top plate, a bottom plate, and the two vertical webs. Bundles or groups of cells form interior columns. Thus, the whole structure consists of various lengths of different combinations of cells. All is needed is the road-surface and the abutments. The amount of automatic welding is so great as to approach 100 per cent. If bridges were constructed on an assembly line, this type of construction is the answer to our problems. I am not willing to say that the ufture may not include prefabricated units somewhat of this nature, especially if we reach a period of extreme steel shortage. Naturally the problems of maintenance and the prevention of corrosion will have to be solved satisfactorily first.

(33) (See Figure 5). The cells I mentioned are clearly visible here and the column size varies with those at the abutment shown at the bottom. The column capitals are rectangular in shape and made of trapezoidal plates. For a 120-foot span the designer suggests a box 5-feet deep at the center line.

(34) This is an end view of a two-hinge arch with tie-girders used as stiffening girders. The span is 250-feet. The arch rib is a fourteen wide flange section—the weight varying according to position. The hangers and top lateral bracing are made of pipes. The tie-girder is stiffened with split pipes, that is, the stiffeners are semi-circular. Likewise the arch rib is stiffened at the hanger connection with pipes split in half. This detail is shown on the right half.

(35) Another suggested solution for a 250-foot bridge is the two hinge tied arch here with the stiffness in the arch rib itself, as can be seen, the arch rib is made up of four pipes—one for each corner. Plates are used for the webs and cover plates. The hangers and tie are 15 WF sections. The detail of the pipe splice is shown to the right. A smaller pipe is shop welded inside the arch rib pipes on one side, then the field erection is managed by slipping the other side

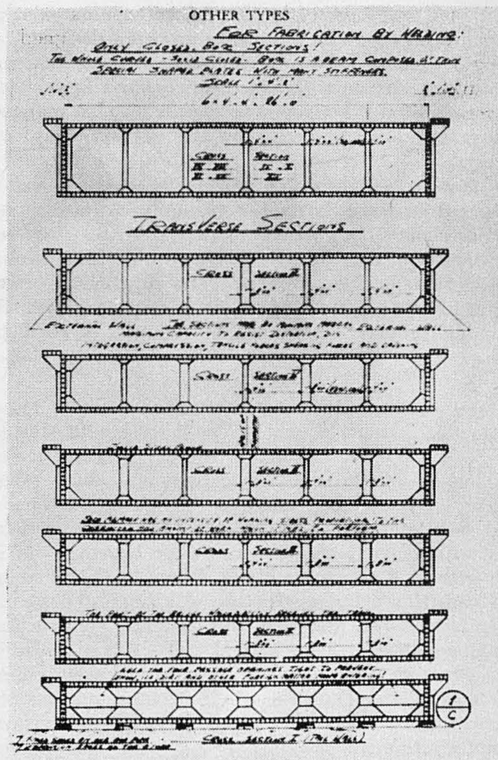

FIGURE 5

over these plugs in the adjoining panel, just like an erector set, the hangers fit inside the section and complete the splice. It is not necessary to weld the pipe immediately since the plugs will keep the section in line.

(36) Mr. Ernst Amstutz of Zurich, Switzerland, suggests this as the design of a 250-foot arch. The two-hinged arch is stiffened with a single semi-circular stiffener located at the center-line of the roadway. This semi-circular plate has a 5-foot radius and serves in such capacity as to eliminate the need of all other longitudinal members, including stringers. The roadway slab spans the distance between the cantilever brackets. The arch ribs are pipes 3 ft. in diameter, with tapered pipes between the two arch ribs being used in lieu of top bracing.

(37) (Se Figure 6). This Vierendeel type of top lateral bracing is becoming more and more common. I agree with many others that it is certainly more attractive. It does require members with connections of large bending moment capacity, but for a welded structure this is relatively simple. In tapering these struts Mr. Amstutz provides a member with bending strength roughly in proportion to the amount of beding moment. Midway between arch ribs the moments and strength being greatly reduced. As you will note this bridge has no tie—the abutments take the longitudinal truss. In the left center of this slide you can see the solution proposed for the intersection between the stiffening girder

nd bottom flange of the transverse bracket. Here is an attempt to reduce the mount of edge preparation necessary and decrease the stress concentrations. These two items are of utmost importance in the design of welded bridges. We eed to give maximum attention to them if we are going to have good cheap tructures.

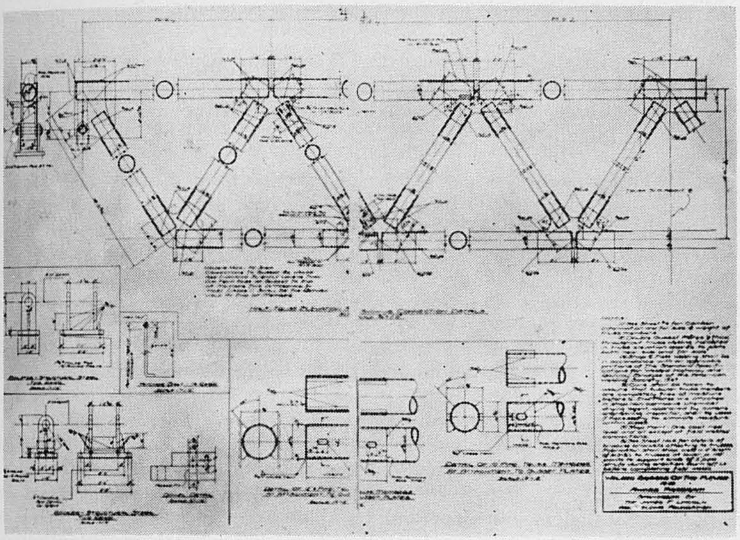

FIGURE 6

(38) Mr. Bleich of New York proposes the expanded 36-inch beam as an rch rib having a depth, after welding, of 48-inches. For arch ribs the shear is ow compared to the high axial load and relatively high bending moment. Thereore, expanding the beam does increase the moment capacity as desired. In order o eliminate top lateral bracing other lateral stability is necessary. It is provided n the rigid hangers connected at the top to the arch rib and at the bottom to he floorbeams wih moment connections. Between hangers lateral rigidity of the ib itself is increased by the use of a bulb beam if available, otherwise the outside dges of the flanges must be increased in area. At the expense of additional welding, channels can be used as flanges with an expanded plate used as the web. f large areas are needed for flanges, the flange could be an I-beam or wide flange urned over in a horizontal position. There is a possibility that should receive erious consideration because it provides large areas and still permits the weld etween the web plate and the web of the rolled section, thus at the weld the lange is not thick.

(39) Temptation was too great to leave the next two slides at home. Here ve see an artist's view of a proposed 990-foot span consisting of continuous memers supported over three spans—270-ft., 450-ft., and 270-ft. At the center pier he longitudinal members are not box girders and elsewhere they are plate girders.

(40) This structure is prestressed. Prestressing steel is not new but it has een used in only a very limited way. Just as prestressed concrete was conceived

and experimented with many years ago and did not come into actuality until quite recently, the existence of prestressed steel bridges is somewhat in the future. However, with more welded bridges being built each year the actuality of a prestressed welded bridge is getting closer. Many studies are being made right now concerning the analysis and probable economy of this type of a structure. At this time it appears that greatest economy will result if the dead-load to live-load ratio is high and if the moment to shear ratio is high. This seems to indicate longer spans of truss and Vierendeel truss type. Arch girders should also be considered.

This bridge has a 36-foot roadway. The main girders are 17′-9″ deep at the piers and 9′-9″ at the center line of the center span, and at the abutments. Composite construction which utilized the slab as a structural part of the steel girder, is used throughout.

(41) This suggested bridge is a welded version of an existing riveted bridge, consisting of three simple spans on a curve. The spans are 32-ft., 78-ft., and 32-ft. approximately. We can see here that channels have been used as the flanges for the four longitudinal girders, and replace the need of forms for the reinforced concrete slab at these locations. Even for such short spans, the completely designed welded structure was estimated to weigh 41-tons as compared to the actual riveted weight of 51-tons.

(42) This is a 330-foot continuous girder bridge having a 28-foot roadway. The individual spans are 100 ft., 130 ft., and 100 ft. The four longitudinal girders consist of tapered plate girders in the vicinity of the center piers and expanded beams for the remaining major lengths in the regions of low shear. A 36 WF 150 is expanded to a depth of 54-inches. At the center pier the plate girders have a depth of 7 ft. 10½ in. Notice the longitudinal stiffener each side of the center pier. Vertical stiffeners are also used on this ⅜ in. web. Not only does the longitudinal stiffener assist in preventing web buckling but being near the compression flange can at the point of maximum moment be effective in resisting bending.

(43) This is a picture of a new bridge across the Rhine River in Cologne, Germany. It replaces one which was destroyed in the last war. The old bridge used 8.500 tons of steel, whereas the new one uses only 5,669 tons. Even with this 30 per cent saving in steel the new bridge has greater capacity than the old one. It was designed by Dr. Ing. F. Leonhardt, and is made up of a series of steel box girders, varied in depth to form shallow, parabolic curves. It was built in 1948. The spans are 404 ft., 605 ft., and 407 ft. The cellular structure becoming common for long span welded bridges has a depth at the center equal to 1/60th of the length. The roadway is 38 ft. wide and there are two 10 ft. sidewalks. It is claimed to be the longest full web continuous girder bridge in the world. A description of this structure can be found in the Transactions of the South African Institution of Civil Engineers, June, 1951.

(44) This is the Pont de Neuilly across the Seine in France. This deck arch bridge has a span of 82 meters. The rib is a curve boxed girder with rectangular frames used as diaphragms. The main reason for showing this slide is to have a reason to discuss the use of a longitudinal stiffener for the vertical webs of the box, even though it cannot be seen on this slide because these longitudinal stiffeners are located on the inside of the box. The stiffener is at about mid-height of the web and consists of an angle with the outstanding leg of each leg welded to the web. Thus, the stiffener becomes a triangular section of extremely high torsional strength as does any closed section stiffener. We should make greater use of closed section stiffeners, especially in welded construction where it is so

economical. Turning angles through 45 degrees so that both legs may be welded to a web or a flange is a simple but very effective means of stiffening.

(45) Australia has built many welded bridges. An excellent article on welded bridges is the one written by Mr. V. Karmalsky, entitled, *Welded Bridges of Department of Main Roads, N.S.W., Australia.* It appeared in *Commonwealth Engineer* in the September, October, November issue of 1949. This article gives a great deal of information on welded bridges including details and observations of experience. Here we see the Hawkesburg River Road Bridge, which was advertised for bids clear back in 1937. This bridge has 14 panels with a total span of 438 ft. It is 65 ft. deep at the center line. The top chord and web members are fabricated H-shapes, but the bottom chord is a fabricated hat shape. To be a little bit more explicit this hat consists of a horizontal plate at the top 16 in. wide. To it are welded two vertical webs 24 in. deep. Thus far we have a channel of unusual shape. However, to the bottom edge of each vertical web is welded an 8 in. plate that extends outward. Thus making the hat shape. There are other cases where such a shape can be efficiently used in welded structures.

My ramblings this morning are not of a nature that can be easily summarized; however, I would like to mention some concluding remarks.

Basically the theory involved in the design of welded bridges is the same as you gentlemen have used for years in the design of riveted structures. The important difference are those of detail. The details of design being the determination of the shape, depth, and width of the overall structure. Next these same factors as applied to individual members in order to put area of steel in its most effective position and still use as little labor of fabrication and erection as possible –keeping the amount of welding to a minimum. To accomplish this end it is essential that we think of the bridge as a welded structure from the very beginning and not pattern it in depth or shape after a riveted structure that we might use for the same span and loading.

But as important, if not more important, than the general shape, depth and width of a structure are the specific details of connections and splices. The cost advantage of weight saved in the principal members can so readily and easily be lost by complicated or expensive detail. With many known exceptions it is still true that simplicity may lead to economy. At least we should investigate the simple solution first and estimate its cost before going on.

The use of closed sections for principal members and closed sections for stiffening elements, that probably are expensive in a riveted structure, may be quite inexpensive in welded structures. However, it should be kept in mind that inaccessible areas will corrode just as fast whether the structure is welded or riveted.

The combination of vertical stiffeners and horizontal stiffeners is not nearly as expensive in a welded structure, and the cost of this combination can frequently be more than offset in the saving of web weight. Sometimes even the flange area can be decreased.

It has been a pleasure to be with you this morning and I thank you all for your very kind attention.