THE APPLICATION OF ELECTRIC ARC WELDING TO THE FABRICATION AND ERECTION OF LOW COST STEEL DECK GIRDER HIGHWAY BRIDGES

THESIS FOR THE DEGREE OF C. E.

WILLIAM BARTLETT SPURRIER

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Bridges

Girders

Electric welding

Seams

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THESIS

One bright spring morning several years ago, the author, then employed by the Michigan State Highway Department, was called into the office of the Bridge Engineer and given his first assignment on field construction. The department had under contract at the time an all steel bridge of the steel deck girder type with steel sheet piling abutments instead of the usual concrete substructure units. An attempt was being made to fabricate special sections of sheet piling by welding together half sections of piling and other structural members in pieces thirty feet long and things were not going well. The steel fabricator had called the office and asked that some one be sent down to see what a H--- of a time he was having and, if possible, make suggestions as to what he could do to prevent the sections from twisting and warping out of shape. Such was the authors first contact with welding and the all steel bridge. The final outcome of this first assignment was the appointment of the author as welding inspector for the State Highway Department.

During his time in direct contact with welders and welding, the author became thoroughly convinced that electric arc welding was the coming thing in the fabrication and erection of structural steel and there were tremendous possibilities for an all steel, all welded bridge. Test specimens with ultimate strengths of 65000 to 75000 lbs. per sq. in. were not unusual and welders who could not produce welds having an ultimate strength of 50000 lbs. per sq. in. of weld metal were not allowed on the job. The all steel bridge, even though it was still in the experimental stage, was a success both from an engineering and a financial standpoint.

This thesis is an outgrowth of the experience the author had in the building of this first all steel bridge. The first portion of the thesis is given over to a detailed study of the process of arc welding and the equipment involved. The strength of welded joints is discussed and data obtained by testing welds to failure included. The proper application of welding to structures in general and the choice of structural shapes most suited to welding is explained and illustrated. Next comes a short discussion of the steel deck girder type of bridge superstructure and the many advantages of this type over concrete beam and girder or steel plate girder or truss construction for short highway spans. The balance of the thesis covers in detail the all steel substructure unit proposed. The advantages and disadvantages and the limitations to be placed upon the choice of this type of substructure are set forth. Many of the difficulties encountered in this type of construction are mentioned and their solution described. And, finally, the cost of the all steel, all welded bridge is compared with the cost of other types using actual cost as tabulated in the Fourteenth Biennial Report of the Michigan State Highway Commissioner as a basis for comparison.

These introductory paragraphs tell in a general way what is included in this thesis and what the author attempts to establish as reasonable conclusions to be drawn from the data submitted.

Welding is defined as a localized consolidation of metals by means of heat, ie., if two pieces of metal are heated to the proper temperature, and then united by pressure, contact, or fusion, they are said to be welded. There are many different processes of welding, but they can all be included in one of two broad classifications which in some cases over-lap, but are nevertheless essentially different. These two general processes are known as the plastic process and the fusion process.

Welding by the plastic process is accomplished by heating the surfaces to be welded until they are in a soft or plastic condition and then forcing them together by external pressure. No additional metal is added to the joint and the surfaces never reach the fluid stage. Under this heading are classified forge welding of all types, electric resistance welding, and the thermit pressure process.

Welding by the fusion process is accomplished by heating the surfaces to be welded to a fluid state and then adding sufficient metal from an outside source to complete the joint. This group includes the simple thermit process, electric arc welding, and gas torch welding which latter two processes are the most adaptable to manual welding.

The field of application of welding by the various methods is being steadily enlarged by research and experimentation and new methods and new equipment are being presented each year by the various manufacturers. Automatic welding machines are now on the market that do their work with as little attention as an automatic screw machine. Welded steel is replacing cast iron in the manufacture of machine parts of all shapes and sizes with a considerable saving in time and expense. Simple structural shapes and plates can be combined quickly and accurately by welding, to take the place of castings, thereby eliminating the costly pattern shop and foundry. In more recent years, welding has been applied to the fabrication and erection of structural steel with considerable success and welding equipment is now considered necessary in an up-to-date fabricating shop. It is not likely that welding will ever entirely supplant riveting, but the application of welding particularly to angular and curved connections is increasing steadily.

From such work is being derived a steadily increasing fund of knowledge and experience which is eliminating fanaticism in the application of welding and it is now recognized that the field for welding is not unlimited. Also it is being learned that each type of welding has it's own field of application in which it is the most efficient and economical. Arc welding is not nearly as successful as gas welding for cast iron and at present, electric welding, is not to be recommended for joints in cast iron where strength is essential. Neither is electric arc welding to be recommended in the case of non-ferrous metals like copper, brass, or aluminum in which oxidation of the parent metal takes place at a temperature far below that of the electric arc. Such welding is not impossible, but it requires a highly skilled operator, and the strength of the joints is not dependable. However, for the welding of structural grade, and the more common alloy steels, no process is superior to electric arc welding in reliability and ease of application. The necessary equipment is inexpensive and highly mobile, and skilled operators are now available in all parts of the country at reasonable wage rates. New types of equipment are eliminating variations in the strength of weld, due to the personal element, and any operator who has received adequate training and who has had a reasonable amount of experience can make welded joints which are uniformly dependable under almost any conditions.

Since the purpose of this thesis is a study of the application of the electric arc process, no further attention will be paid to any of the other methods of welding.

It is a well known fact that, given the right type of current supply, an electric arc can be formed between two pieces of metal - known as electrodes - and that, as long as the supply of electric current is ample for the length of arc held, the arc can be maintained. The gap between the two electrodes is bridged by a stream of electrons which complete the electric circuit. The resistance offered by the surfaces of the electrodes to the passage of these electrons results in intense heat at the surface of the electrode, and a crater of molten metal is formed. If we substitute a steel wire of proper size for one of the plates, we can raise the temperature of the surface of the plate and of the end of the wire to the stage of fluidity, and the metal from the end of the wire will flow on to and unite with the molten metal at the surface of the plate. By proper manipulation of the wire electrode, a uniform deposit of new metal can be placed upon the surface of the plate in the form of a ridge, this deposit being known as a "bead". Now, if we go a step further and proceed to draw the arc at the line of intersection of two plates, the surfaces of both can be raised to the fluid stage, and the bead of new metal from the wire electrode placed at the intersection. The metal from all three sources is thus fused into one common mass and in this way the two plates are joined together and the joint reinforced by the addition of new metal from the wire.

The equipment required for arc welding is not particularly complicated or expensive, and usually consists of the following pieces of apparatus:- a welding generator with a suitable prime mover, which may be either an induction motor or a gas engine, together with the necessary appurtenances for the control and measurement of the flow of current to the arc, an electrode holder suitable for use with the ordinary commercial electrodes and the necessary cable to establish connection with the generator, a ground clamp and cable, suitable protective devices for the operator, most important of which is the welding helmet with its specially made dark glass eyepiece, and a supply of electrodes. All of this equipment is made by a number of different concerns and no attempt will be made to discuss the relative merits of the products of these different companies. However, each of these pieces of equipment will be described briefly with the idea of giving the reader of this thesis some idea of the characteristics of the equipment, and the conditions which must be met by the manufacturer.

A steady, uniform flow of electric current to the arc is one of the essentials for reliable and economical welding, and the manufacturers of welding generators have given much attention to the details of these machines and their controls. A good welding generator must deliver current at the proper voltage and amperage for the work at hand, must not be overly sensitive to variations in the resistance of the electric circuit which it supplies, and must be able to withstand repeated short circuits. This sounds like a rather large order for a generator, but the modern welding generator will meet all these requirements with ease and many others besides.

The present day generator is either a two or four pole compound wound machine of the "bucking field" type. The term "bucking field" means that by some means or other there is set up within the machine a field opposite in direction to the main field, and it is this opposing or "bucking" effect that makes possible high current flow at low voltages and permits adequate control. There are many variations in the way in which this bucking field is produced and used. One company uses a series of interpoles. Another uses a split pole piece with windings on each leg. Still another uses a pair of interpoles with tapped windings. The object of all these manufacturers is to obtain a series of amperage ranges without changing the value of the main field appreciably and without moving the brushes from the neutral point at which commutation is the best. In all cases, fine adjustment of amperage produced is obtained by very limited variations in field excitation. The final result is a generator whose output of electricity at low voltage and high amperage can be controlled by the combination of a multi-point switch and a field rheostat.

Welding generators are rated by their nominal capacity in amperes at full load and range from 100 amps which is the smallest to 600 amps or more in steps of 100 amps or thereabouts. Generators rated at either 200 or 400 amps are the most common and will meet most requirements. The 200 amp machine will supply a current range from 50 to 300 amps depending upon the voltage and length of the arc. The 400 amp machine will supply from 75 to 500 amps and is of ample capacity for almost any type of welding using either bare or coated rod. The smaller machine is hardly large enough for steady use with coated rods.

The prime mover for welding generators may be either an induction motor or a gas engine. Where adequate AC current is available, an induction motor is by far the best driving unit and the most economical. A motor rated at 20 H.P. with a continuous load temperature rise of 40 C. will drive a 400 amp generator and a motor of half that size will drive the 200 amp generator. For field welding, the gas engine driven unit is by far the best and in many cases is the only type that may be used. A 400 amp generator will require a 50 H.P., six cylinder engine of the heavy duty type running at 1500 rpm., or more under full load. A 200 amp generator will require a 30 H.P. engine, also designed for full load at 1500 rpm, or more, which may be of four cylinders. Either engine will require a reliable governor to protect the engine from over running when the load is suddenly removed as it is whenever the arc is broken and also to bring the generator up to speed quickly after the arc is struck again.

As has been said before, the controls for the ordinary welding generator consist of a multi-point switch of some type to vary the welding current in steps of from 50 to 100 amperes and a field rheostat for finer adjustment. This arrangement permits close and adequate control of the arc and also allows some variation in the relation of current to voltage, this being important when extremely long leads are required to reach the work. Modern day sets also have a reactance in series with the welding leads to stabilize the arc and to prevent the surge that always tends to occur when an electric circuit is shorted. An ammeter and a voltmeter are standard equipment on all welders and are necessary for the proper setting of the controls for any particular piece of work. An experienced welder will experiment on scrap material until he has the amperage and voltage at the arc exactly as he wants it for the work at hand, and, once set, expects the machine to be stable enough to maintain that setting without further adjustment or attention.

Normally the details of the structure will, to a large extent, determine the arrangement of the parts and the location of joints. The same care is necessary for welding as for riveting to avoid inaccessible joints and the detailer should be familiar enough with the possibilities of welding to know when and how to apply welding. There is never an excuse for an inaccessible joint. In general, work should be arranged in such a manner that short leads are possible and a minimum movement of the welding machine necessary.

One of the leads from the welding machine is connected to the work by means of a "ground" clamp which can be securely fastened at some point where it will not be in the way of future work. So long as there is a continuous metallic circuit offered by the work, the ground clamp seldom, if ever, has to be moved, and is often fastened in place by a short tack weld.

It is necessary for the welder to protect himself carefully against the possibility of arc burn caused by the large proportion of ultra-violet rays in the flame of the arc. All exposed surfaces of the body <u>must</u> be covered. Ordinary heavy clothes are sufficient for the main portions of the body and canvas gloves will protect the hands. The eyes and face are usually protected by a hood or helmet with eyepieces of specially prepared dark glass which filters out most of the dangerous rays. The effect of arc burn is the same as sunburn, but is more intense and dangerous. Burned eyes are just about the most painful things that a human being has to experience, and there is little or no relief to be given the patient. He is temporarily blind and a mild local anaesthetic is all that can be applied to the eyes. Natural healing processes will heal the burn in 48 hours if it is not too severe, but medical attendance is always necessary at first. No person should watch the play of an electric arc at a distance of less than fifty feet without some eye protection and, if it is at any time necessary to work close to a welder, suitable protective glasses should be worn at all times.

The wire electrode, which has been mentioned from time to time, is in most cases, a straight piece of wire or rod 14" long, either bare or coated with a special paste or flux. Standard sizes are expressed in terms of the nominal diameter of the metallic portion of the electrode and are 3/32, 1/8, 5/32, 3/16, 1/4 and 3/8 inches. A heavy coating of flux may double the actual diameter of the finished electrode, but the designation of size is always the diameter of the metal core. The metal in this core is a low carbon steel of better than average grade and of such analysis that the metal deposited by the arc is practically the same in composition as the parent metal. The American Welding Society has standardized the analysis of the rod to be used for various types of welding and their Specification E-I-B for rods to be used on structural grade steel is as follows:

Carbon	.13 to .18%
Manganese	.40 to .60%
Phosphorus	less than .04%
Sulphur	less than .04%
Silicon	trace

This steel should have an ultimate tensile strength of from 55,000 to 60,000 lbs., per sq. in., with an elongation of 10 to 12 per cent. When used by the average welder, the metal deposited will have an ultimate tensile strength of somewhat more than 40,000 lbs., per sq. in., and really fine welders will raise this figure 10,000 lbs. It is very seldom however, that any welder using bare wire will make a joint stronger than the parent metal and this must be taken into consideration when specifying the type of rod to be used for any piece of work.

Most manufacturers of welding rods also make what is known as a flux coated rod. The purpose of this flux coating is to produce a reducing atmosphere, usually carbon monoxide, which will envelop the arc flame and prevent the formation of oxides and nitrides by reaction with the oxygen and nitrogen of the atmosphere. The molten metal in the arc crater has an affinity for oxygen and nitrogen, and if the arc is maintained in an atmosphere composed chiefly of these two elements, oxides and nitrides are formed within the deposited metal and seriously impair its strength. Weld metal deposited with a coated rod is normally much finer in texture than metal deposited with bare wire and normally has the fine silky appearance which is typical of structural steel.

One foreign manufacturer makes a rod that is coated with spirally wound asbestos yarn which is impregnated with chemicals and serves the same purpose as the coating of domestic rods. Laid parallel to the core and within the coating is a fine aluminum wire which serves as a cleansing agent for the weld metal and has a tendency to float the oxides and nitrides to the top of the weld where they are absorbed by the slag. This rod will deposit weld metal which compares very favorably in strength with that deposited by the better grades of domestic rods.

The steel core used in these coated rods is exactly the same analysis as bare wire and has the same physical characteristics so the increase in the strength of the weld metal is evidently due to the effect of the coating. More amperage is used with coated electrodes than with bare electrodes and, as a result, a larger, more powerful welding machine is required. Also the cost of coated electrodes is nearly double that of bare rods, but if there is any need for dependable strength, the coated rods are the thing. This increase in cost is offset to some extent by the fact that an experienced welder can work much faster with the coated rods. Another thing worthy of note in discussing coated electrodes, is that the coating is a non-conductor of electricity and thus eliminates what is known as side arc, ie., the formation of the arc from any part of the rod except the tip. This is of very great importance to the welder when working in close quarters where it is almost impossible to avoid touching other parts of the structure in an attempt to reach the joint with the end of the rod. It is possible with a coated rod to lay the rod on a grounded steel plate, start the arc at the exposed end, and then watch the arc deposit a semblance of a bead on the surface of the plate without further attention. This is a very pretty trick to watch and might have some possibilities.

So much for the equipment and theory.

After the work is all set up and all the necessary equipment correctly arranged and adjusted, there is nothing left to do but strike the arc and go to work; at least that is the impression some vendors of welding equipment seem to leave with their customers. Therefore, let it be said now, that good welding is an art in itself. Striking and holding the arc is a knack that requires much practice. After the arc is established, it must be held to the proper length and manipulated in the proper way to obtain the desired results. However, skill in arc welding is not beyond capabilities of any conscientious workman.

As has been said before, striking the arc surely and quickly is a trick that requires much practice and experience. The end of the rod is touched to the work with a slightly stroking motion of the end and then drawn back about an 1/8 inch. If the rod is not drawn away quickly enough, it "freezes" to the work and has to be broken away by main strength. If drawn away too quickly and too far, the arc is broken and the whole process has to be started again. Skilled workman develop the ability to strike an arc by direct touch without the stroking motion and this is often necessary when working in cramped quarters.

After the arc is once started, it is necessary that the length of arc be maintained uniform in length and as short as is reasonably possible. An eighth of an inch or thereabouts is the proper length, and any great variation from this length will effect the strength of the weld, usually in an adverse manner. Holding an arc of correct length is also complicated by the necessity of "feeding" the rod into the arc as the end of the rod melts off and is deposited. Fortunately, holding an arc of correct length becomes habitual after a reasonable amount of experience and practice, just as swinging a golf club or a baseball bat correctly becomes a matter of habit rathen than of conscious effort for the experienced player. This is indeed fortunate for the welder cannot see the end of the rod after he has pulled his helmet down over his eyes, and therefore has to depend entirely on a sort of "sixth sense" to determine whether he is holding a short arc. To the bystander, the crackle of an arc of correct length is very distinctive, and, once heard, easily recognizable afterwards. If the arc is too long, small explosions occur within the arc at roughly uniform intervals and the popping sound of these explosions is an immediate indication to a trained inspector that a long arc is being used.

Variations in the length of arc have a very direct bearing on the strength of weld. The length of the actual flame of the arc is some where near uniform for any combination of rod size and amperage that is likely to be used. Therefore, if the end of the rod is held close to the surface of the parent metal, the flame penetrates deeply, a crater of molten metal is formed, and deep and thorough fusion takes place. If too long an arc is held, the arc flame plays on the surface of the parent metal, but does not raise the surface to the melting point, and, as a result, no fusion takes place, for the presence of a crater of molten metal is essential for fusion. This word "penetration" is ever present in a discussion of welding and welding methods for the success of a weld depends entirely on the amount of penetration secured. When welding two plates together, it is absolutely essential that the intersection of the plates be firmly fused together and it is for evidence of the existence or non-existence of this fusion that the inspector seeks first. One characteristic of almost all poor welds that the author has seen broken was lack of bond at the line of intersection of the connected parts. Most good welders use what is known as a "stringer bead" placed at the intersection line to insure proper fusion. This stringer bead is nothing but a small bead carefully placed at the intersection to insure proper bond between pieces.

The finished weld is built up of a number of separate beads, each firmly bonded to the others, and the adjacent parent metal. The number of beads used varies with the size of the weld to make, the size of rod used, and, to some extent, the personal likes and dislikes of the welder. Where a very large weld is to be made, it is customary to place the stringer beads with a small rod and finish with a large rod which will deposit metal more rapidly. The author has met one welder who had the ability to place a small bead with a 1/4" rod and preferred to do all his work with this size. Normally a 3/8" triangular fillet weld which is a common size is made by first placing a small stringer bead with say an 1/8" rod and then finishing with a 3/16" or 1/4" rod using a total of two or three beads or, as they are more commonly called, "passes." A butt and bond weld of two 3/8" thick plates to the top of an I-beam with a gap of 1/2" between plates will require three passes - a stringer bead at each bottom corner of the gap and one large bead to fill the gap to the surface of the plates.

The weld metal in large beads is placed by a sort of weaving motion of the rod which causes the end of the rod to move in the form of a series of over-lapping triangles or ovals, depending on the size of the bead to be placed. It is the ability to, in this way, place the weld metal in a smooth and even ridge with thorough fusion between subsequent beads that makes the finished welder. If one of the pieces connected is thicker than the other, it is necessary to play the flame of the arc longer on the thicker piece so that both pieces are raised to the temperature of fusion. If this is not done, there will be no bond to the thicker piece.

The author's observation of welders and their work had lead him to believe that the knack of welding is a thing that some workmen can never acquire. Steady hands and nerves are essential, and, lacking these, the workman will never be able to hold a short, steady arc for hours at a time. A good welder must be conscientious about his work, patient, and ambitious, for welding can not be learned in a day and many are the trials and tribulations of an apprentice welder. He must also be watchful of his work to avoid the formation of bad habits and the adoption of faulty methods which in later years will have a bad effect on his work. This latter is of particular importance for good welding is largely a matter of habit.

The fact that good welding is a habit is the one thing that is now and will continue to build up confidence in welding as a method of erection. Many engineers feel that because the personal element is unavoidable in welding, and because there is no non-destructive test of a finished weld, welding and welders can never be relied on as we rely on properly inspected riveted construction. Their position was perhaps well taken during the early stages of welding when all work of this kind was more or less experimental, but that day is past. One particular welder worked almost continually for two weeks on a research problem supervised by the author, and his welds averaged better than 40,000 lbs. per sq. in., ultimate tensile strength. This was with bare wire. The author was also responsible for the qualification of welders for the Michigan State Highway Department for two years, and the average of the strengths of the test samples submitted by some twelve or fifteen welders was better than 55,000 lbs. per sq. in., ultimate tensile strength. This latter group of welds were made with coated rods of one kind or another. To the author, this is not only an overwhelming argument for the use of coated rods, but also for the reliability of such welding. These averages represent a safety factor of 4 and 5 respectively on the basis of the design strengths now in common use. This superiority of coated rod welding over bare wire welding became so obvious to this department that they wrote into their specifications for welding the requirement that <u>all</u> welding shall be done with a coated rod of some approved type.

Any method of setting up work that is satisfactory for riveted construction will be suitable for welding and in many cases simpler methods may be used. If the parts to be handled are small and light in weight, steel C-clamps can be used to advantage. Heavier pieces will require erection bolts but these can be limited to the number necessary to carry the dead load of the part at a high unit stress in the bolts. Perch angles can be tack welded in place either in the shop or in the field and members supported in that way until the finished welds are made, at which time the perch angles can be knocked off with a sledge. It is often possible to hold the piece in place with the erection derrick and a few C-clamps until sufficient welds are made to carry the dead load. This is the most economical and satisfactory way and a skillful detailer can so design the parts as to make this possible in the majority of cases.

The difficulties encountered in welding are none of them serious and none of them in any way unsurmountable. First, in order of importance, is the position of the welder in relation to the work. No man can work well when in a cramped or otherwise uncomfortable position when his work requires steadiness of hand. For this reason it is always advisable to provide the welder with a platform, staging, or whatever else is required for his comfort, built in such a way that he can both reach and see his work easily. If such platform or staging can be built in such a way as to allow him the unrestricted use of both hands, so much the better.

Overhead welding is not very nice work for an inexperienced welder, and some inspectors will tell you that one sure way of checking up on a new welder is to try him on overhead work. If he finishes the assignment without burning himself he must be a fair workman. The main difficulty with overhead work is the shower of sparks and molten slag that fall from the arc and this shower of hot sparks is much worse if the welder can not hold a short and steady arc. Some welders will also make the statement that it is impossible to do overhead welding with a heavily coated rod but a little observation of the work of a skilled welder will soon prove the falsehood of such a statement. However, it is always wiser to eliminate overhead work if possible, and a competent detailer can usually find the way out of such cases as are encountered.

Joints inaccessible to the welder are a direct indictment of the detailer who made the working drawings. No draftsman assigned to such work should be ignorant of the process of welding and the equipment used and certainly no one is to blame but himself if the erection crew reports an inaccessible weld. This latter statement holds true for both riveting and welding. However, it might again be wise to call to attention the fact that the insulation on a coated rod makes it possible to work in places that could not be welded with bare rod.

Difficulties due to the fit of the parts to be welded are easily avoidable by correct detailing and careful inspection. When two surfaces are to be welded together, they should meet smoothly, and, if necessary, be clamped or bolted together until sufficient welding is done to hold the pieces in place. If the pieces meet at an angle there should be contact along the entire length of the line of intersection. If the pieces lap or lie in the same plane, the adjoining surfaces should be parallel and fit tightly together.

Sometimes when making butt and bond or some types of fillet welds, insufficient clearance is allowed between surfaces, making it impossible for the arc to reach the line of intersection and bond the lower corner properly. This difficulty can be eliminated by careful detailing and inspection. Experience and some research work by the author indicates that the minimum gap between plates of a butt and bond weld should be the thickness of the thickest plate plus 1/8" and that little or no bond to the supporting structure can be expected if this allowance is not made. It is possible of course to allow a much wider gap, but there is no economy in so doing, for the time and material required increase much more rapidly than the strength of the weld.

Undercut or rounded edges on the plates will cause a tendency towards weak welds and should be avoided if possible. Even the slightly rounded edge of a universal mill plate will make bonding the lower corners of the weld difficult, particularly if bare rod is used. Some of this difficulty can be eliminated by the use of coated rod in which the added heat and penetration makes proper fusion possible. The only real cure for this trouble is the beveling of the edges of the pieces to be connected in such a way as to leave a wide open Vee into which the weld metal can be placed.

A 60 or 90 degree Vee with a gap of 1/16" or slightly more between the bottom edges of the plates is the best possible preparation for welding, but this is not always obtainable for economic reasons. No quick, accurate and economical method of beveling thin plates has yet been developed and at present, bevel cutting with an oxy-acetylene torch is the best method. This is slow at best and leaves a film of oxide on the plate which is very hard to remove and which has a detrimental effect on the weld. Beveling with a milling machine is out of the question on all fabrication jobs because of the time and expense involved. However, it has been proved conclusively that welds as strong or stronger than the plates connected can be made without beveling the plates, so this particular difficulty is more a matter of theory than of practice.

One other difficulty encountered, particularly in fillet welding, is lack of support for the bead. The welder is, after all, dealing with a liquid metal which obeys all the laws of gravity and, if not supported, will flow over the edge of the plate. No welder can place a 3/8" fillet weld on a projection of only 1/4" and make it look right. The remedy for this difficulty is obvious.

Difficulties due to expansion and contraction of parts are the most troublesome and by far the hardest to deal with. Careful design of the component parts of the structure based on a practical knowledge of welding practice is the best remedy. Long continuous beads are always a cause of warping and should be avoided wherever possible. The allowable unit stresses for welds now recommended by the American Welding Society are very conservative, and any good welder can deliver welds with a factor of safety of 4 or 5 based on these allowable stresses. Therefore, the writer believes that it is both logical and safe to design welded joints in the same way riveted joints are designed, ie., divide the stress to be carried thru the joint by the strength of each unit of joint material. Stitch welding by which is meant welding short beads with unwelded portions between adjacent beads has been proved to be practical by actual tests of full size joints and a weld three inch-skip three inch joint will develop an amazing strength in longitudinal shear or pure tension.

In this way it is possible in most cases to avoid calling for a long continuous bead.

If it is necessary to make a long continuous welded joint, some of the expansion can be overcome by what is known to welders as "back stepping." This consists of making the joint in a series of short welds by putting in say 6" of weld and then moving ahead a foot and putting in another 6" and finally going back the full length of the joint and filling in the unwelded portions to complete the joint. This way no one portion of the connected parts has a chance to become greatly hotter than the coolest portion, and the heated portions have a chance to cool between passes.

When welding a joint between two fairly long members there is always a tendency for the far ends to crowd together, thus spoiling the alignment of parts and, in the case of butt and bond welds, closing the gap between the members. This can be controlled by means of small wedges or by tack welding. Tack welding skillfully applied is one of the most useful kinds in welding. Short bits of weld can be used instead of erection bolts, thus eliminating the punching of holes in the shop. Light parts can be held together by C-clamps while tack welds are made and the clamps then removed for use elsewhere. It is even possible in some cases for a helper to hold the parts together while the welder puts in tacks. By means of tack welds, entire structures can be set up and the fit of joints checked and adjusted before any final welding is done. Expansion and contraction can be checked or controlled by tack welds. In short, the opportunities for simplification of erection by properly applied tack welding is almost limitless.

If undue distortion does take place it can usually be remedied by cold working or the application of heat and quenching at the proper points. This latter method is very tricky, for there is always the danger of releasing some internal stress, retained within the metal after rolling, which will make the deformation worse instead of better. For this reason, if no other, it is always wise to avoid long continuous welds. It also should not be forgotten that there are many instances in which welding is <u>not</u> the practical way to make the joint. The writer witnessed one attempt to use welding in the fabrication of special sections of steel sheet piling that convinced him thoroughly that using beads 20 to 30 feet long to connect comparatively light members was neither practical or economical.

Inspection of welds is still much a matter of routine. For the fabrication and erection of structures there has not yet been discovered a non-destructive method of testing that can be applied out on the job. X ray or gamma ray pictures will show in detail the inner structure of the deposited metal, but no one would think of taking the necessary apparata out to a bridge site and attempting to take pictures of every joint in the bridge. There has been some experimental work done in the study of welded joints in much the same way in which riveted joints are tested. The investigators believed that it would be possible to listen to the sound transmitted thru a joint by means of an ordinary physicians stethescope and in this way spot gas pockets, oxide inclusions and other things that tend towards weakness in a welded joint. A group of fairly capable welding inspectors were given some instruction in this method and then asked to pass judgement on a series of specimens. Comparison of their judgements with actual strength tests on the same specimens showed that the results of this type of inspection were erratic and unreliable and that, without the aid of apparatus of any kind, this group of inspectors could do better work than with the stethescope. This does not mean for one moment that this method may not be developed into something very exact and usuable. Nothing would be more acceptable to both the inspectors and the welders themselves. The average welder seems to take great interest in his ability to make good joints and obviously wants to prove that he is as good a workman as any other welder on the job.

For closed vessels, soap bubble or oil tests have become standard and the results obtained check very closely with break down tests on the same vessel. Many of the old time steel erectors who are still hostile to welding, occasionally try the good old "sledge hammer test" and many are the surprises that occur. On one job a riveter was caught trying to knock off some welded perch angles with a 20 lb. maul. As punishment for this offense, he was instructed to keep at it until he broke off one or two of the angles which he had damaged. He was a convert to the use of welding before his job was finished and was also unable to lift the sledge the next day. The writer also saw some welded special sheet pile sections which had been pounded down to refusal with a 2000 lb. gravity hammer at a 12 ft. drop in which the body metal had failed without damaging the welded joints in any way.

The use of welding as a means of fabrication and erection does have a very definite effect on the choice of sections to be used. Just as in riveted construction, the shape of and method of making the necessary joints is often the controlling factor, but this is less true in welded structures than in riveted structures, as a great part of the economy possible thru welding is due to the elimination of parts which, in riveted construction, are used only as a means of connection. A typical case is that of the stiffener angles in plate girder construction.

From a stress standpoint, the outstanding leg of a stiffener angle is the portion designed to carry the load, and the other leg serves only as a means of attaching the angle to the web of the girder. Also if a stiffener angle is to be effective in transmitting stress from the ~~bottom~~ flange to the web, it must be milled and ground to an exact fit against the flange. Welded construction eliminates entirely the use of an angle section and substitutes a plate of the proper size and thickness, thus reducing the weight of material required one half. Instead of grinding the plate to an exact fit against the ~~bottom~~ flange, it is cut a trifle short and any variations in fit taken up by the welding. And the bearing area is always 100 per cent of the sections.

Another typical case is the splicing of the top and bottom chord section of trusses. Riveted construction requires the use of additional splice members and increasing the number of rivets at the joint to develop the entire section. Welded construction requires only that the joint be fully welded with, in case the stress at the joint is heavy, a small patch plate welded along its entire perimeter.

In general, the choice of section for welded construction is based first upon its adequacy to carry the stress imposed and secondarily upon the way in which the joint may be made the simplest. All elements not necessary for the transmissal of stress are eliminated wherever possible. There must be, however, enough perimeter to allow the amount of welding required to transfer the stress across the joint. The ideal condition is attained when there are large areas in contact, thus providing the maximum perimeter for welding. Next best is for the two sections to butt against each other so that splice plates may be used. Least desirable is a joint which requires a gusset plate. The economies obtained by the use of welding are not in the cost of making the joint, but in the reduction of the amount of material required, and the elimination, in so far as it is possible, of all punching and machining. Modernization of cutting methods and the general adoption of the oxy-acetylene torch as a method of cutting and shaping parts has added greatly to the ease with which members can be fitted for welding.

Plate girders have heretofore consisted of a web plate, two or more cover plates, top and bottom angles with or without side plates, stiffener angles, and splice angles and plates if needed. Of these parts, only the web plate, the outstanding legs of the top and bottom angles and the outstanding legs of the stiffener angles are used in welded construction. A welded plate girder consists of a web plate with two heavy plates welded to the edges and stiffener plates welded to both the web and the top and bottom plates. In this way the required flange area is concentrated as far as possible from the neutral axis and attains a maximum of effectiveness. Side plates are not used and, if one plate will not suffice for the flange cover plates are added. As has been said before, the connecting leg of the stiffener angles is eliminated entirely. The girder is designed exactly as for riveting and the amount of welding proportioned in the same manner as rivet spacing. Flange splicing consists of butt welding the plates together and perhaps adding a short splice plate and web splicing is done in the same manner.

That such a girder is strong and most efficient is borne out by tests made by the Westinghouse Electric Company on three similar beams. Beam 1 was the usual riveted type and weighed 798 lbs. When tested to failure by center loading failure took place by buckling of the top flange, the yield point being 55000 lbs., and the ultimate load 68900 lbs. The section modulus was 61.1. Beam 2 was identical with Beam 1 except that the parts were welded together and the section modulus 60.2. This beam failed by crimping of the top flange, the yield point being 65000 lbs., and the ultimate load 77200 lbs. The bending moment was 27000 ft. lbs., and the maximum fibre stress 53800 lbs. per sq. in. Beam 3 was designed strictly to take advantage of all the possibilities of welding and weighed only 656 lbs. Under test, the beam failed by buckling of the top flange, the yield point being 65000 lbs. and the ultimate load 78000 lbs. Although the weight was less, the section modulus of this beam was 62.2 and the maximum fibre stress attained was 52700 lbs. per sq. in. Thus it is shown that although the welded beam weighed 16 per cent less than the riveted beam, it was considerably stronger. And failure was not due to weakness at the welds.

The individual welds used to connect the component parts of a welded structure normally fall within one or another of the following classifications which are in common use among welders and designers:

- A Triangular or Fillet
- B Butt
- C Butt and Bond
- D Plug or Rivet Welds
- E Combination Plug and Butt and Bond Welds

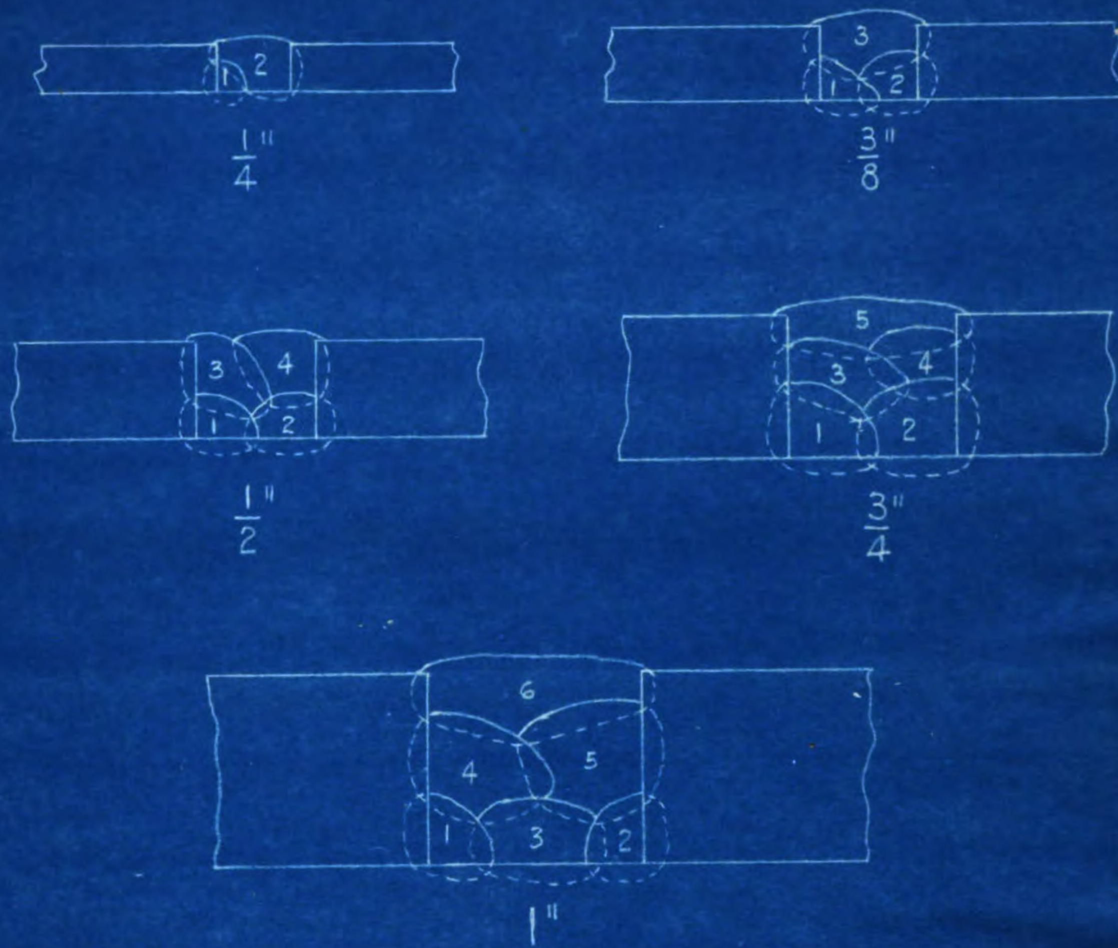
Triangular or fillet welds are, as the name indicates, roughly triangular in form and are used principally where the parts to be connected either overlap or meet at right angles. The stress to be transmitted is usually shear, although in some cases this type of weld is used in tension. Small fillet welds can be made with but one pass, but this method should be limited to not cover $\frac{1}{4}$ inch depth of throat. For larger welds between thick plates two or more passes should be used arranged as shown on Plate I.

Butt welds are used to connect plates which have their edges "butted together" and the stress to be transmitted is usually tension. Actually the pieces should not come in contact with each other as some allowance has to be made for the addition of weld metal. Good practice includes the beveling of the abutting edges, but this is an expensive process and is seldom used except for pipe welding or the connection of small sections. The number and sequence of passes for the welding of plates of various thicknesses is shown on Plate I.

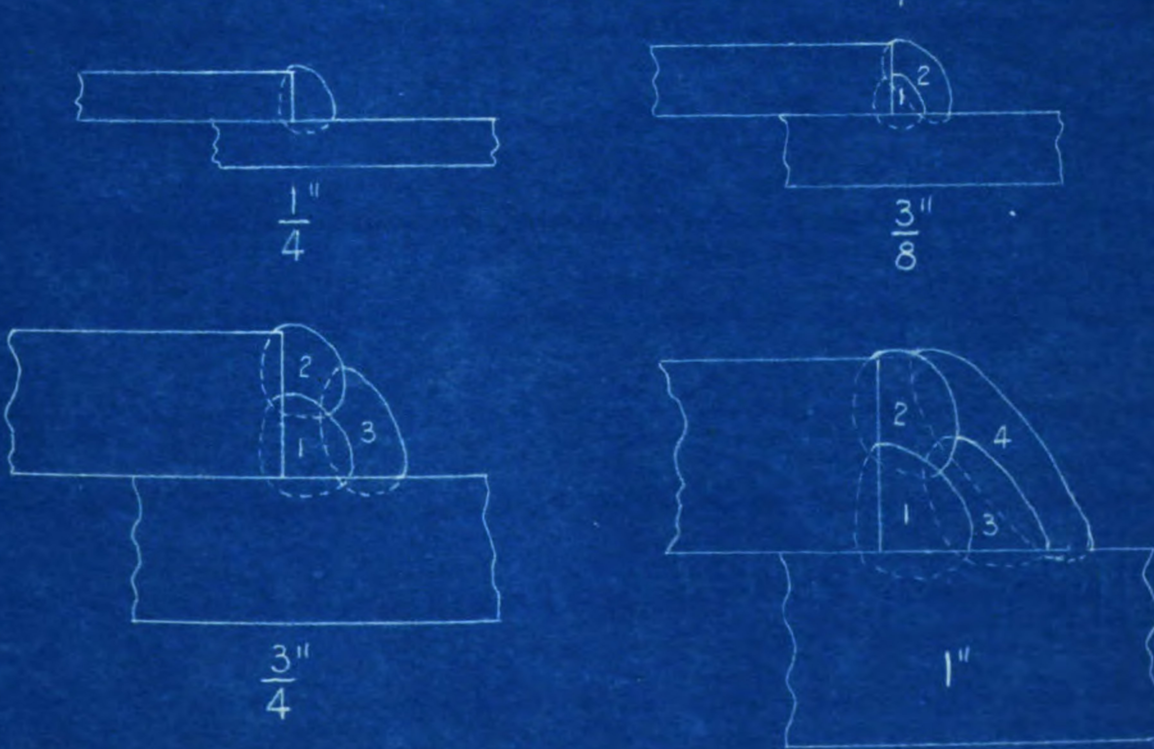
Butt and bond welds are, as the name indicates, used to connect two pieces and at the same time, bond both to a third, and is the weld used in the so-called "battle-deck construction." A gap is left between the top plates and the weld made by filling this gap with weld metal. This is also a means of avoiding the beveling of plates mentioned in the paragraph above for, if the gap left is of proper width, it is possible to make a good joint between square edged plates. This joint should always be made with two or more passes as the strength of the weld depends largely on the stringer bead placed at the intersection angles. The number and sequence of passes to be used is shown on Plate I and is the same as for simple butt welds.

METHOD OF WELDING JOINTS

VARIOUS THICKNESSES OF METAL



BUTT WELDS



Plug welds are the counterparts of rivets and are used in a similar manner. One of the pieces to be connected is punched and then welded to the other by filling the open hole with weld metal, taking particular care to bond the pieces at their line of contact. So far as the actual welding is concerned plug welds are a variety of butt and bond weld and the number and sequence of passes shown on Plate I for butt and bond welds will apply. Tests have shown that plug welds less than 13/16 inch in diameter are not successful and that 15/16 inch diameter is a better size. The added welding in a 17/16 inch diameter plug does not increase the strength in proportion and therefore the larger plug welds are uneconomical.

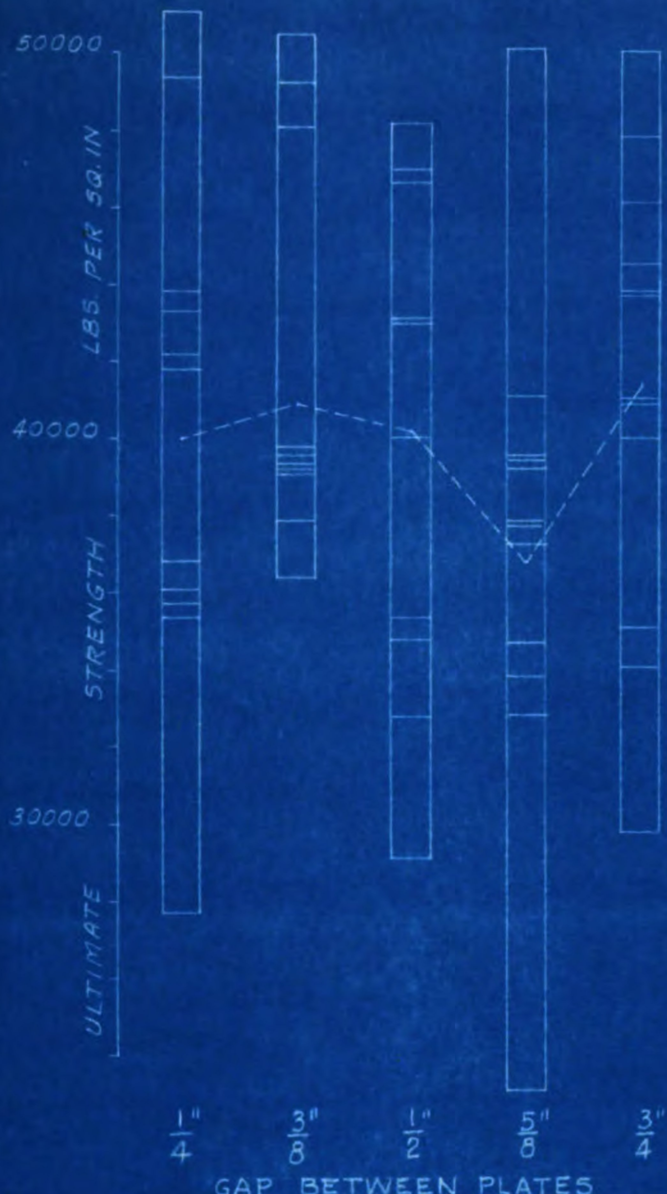
It is possible to combine plug and butt and bond welds and this type of weld was studied in detail by the Highway Dept. The plates are prepared for welding by first punching a series of holes of say 15/16 inch diameter in a straight line and then cutting the plate along a line through the centers of these open holes. The result therefore is to increase the length of weld possible for a plate of any given width by making the edge of the plate serrated as is the perforated edge of a postage stamp. This however is an expensive process and tests indicate that the strength of such a weld will actually be less than strength of a properly made butt and bond weld.

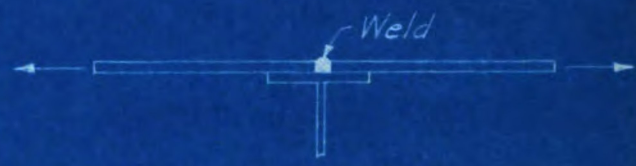
The strength of welded joints has been the subject of many investigations, and many papers have been written on the subject. Therefore, the discussion in this thesis will be limited to a report of the findings of an investigation made by the author for the Michigan State Highway Department. This investigation dealt mainly with the butt and bond weld and the possible effect of position of the weld upon its strength. It must be borne in mind in studying these findings that all welding was done with bare wire and that the strengths obtained are therefore low. Subsequent testing of welds made with coated electrodes proved beyond doubt that a strength of 55000 lbs. per sq. in., in tension, is easily obtainable and that good welders will normally make welds stronger than the parent metal. The Highway Department required that all welders be pre-qualified and test samples breaking at 70000 lbs. per sq. in. were not unusual. In fact it became necessary to reduce the net width across the weld to prevent failure of the test specimen through the parent metal which after all proved only that the weld was stronger than the parent metal. The following plates and the explanations thereon are from the original report.

One problem which is easily solved by welding but very difficult in riveted construction is that of joints in a skewed structure. Normally all joints in a steel structure are at right angles for the accurate bending of an odd shaped connection plate to a predetermined angle of skew is entirely too costly for general practice. Also there is a limit to the ability of steel

BUTT & BOND WELDS

PLATE A

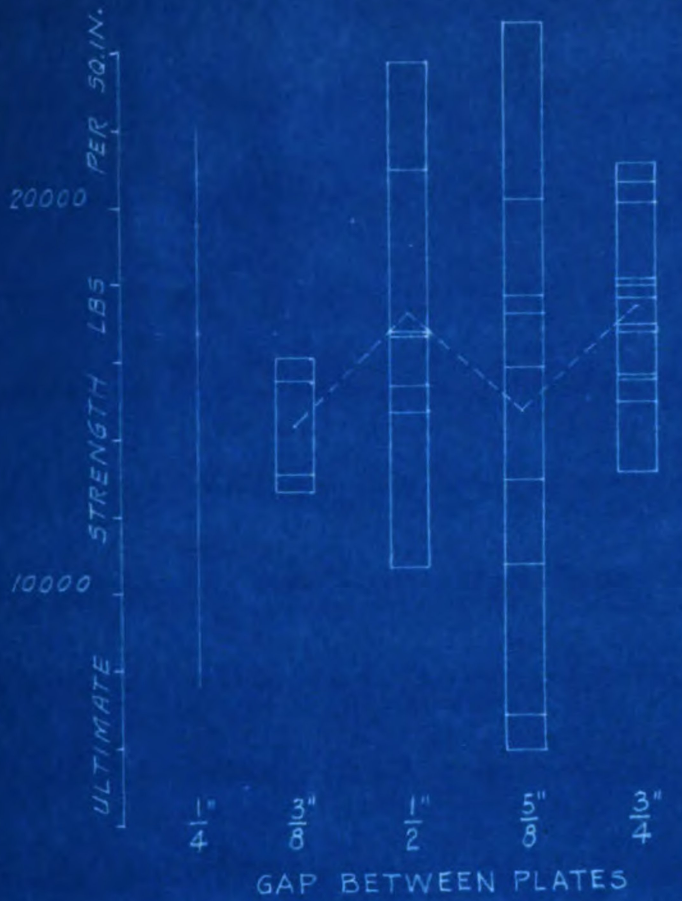




This plate shows the effect of the gap between the plates upon the strength of a simple butt and bond weld stressed in tension parallel to the plates. With the exception of the welds with a 5/8" gap the strengths are remarkably uniform due perhaps to the fact that the weld failed in all cases. Why the 5/8" gap welds were so weak is hard to explain. One possibility is that there is, for each combination of plate thicknesses, one particular gap in which the concentration of heat in the weld metal reaches a maximum and results in an undue amount of oxidation. Welds in a narrow gap are quickly made and the metal does not absorb a great amount of heat. Welds in a wide gap are made slowly and, altho a great amount of heat is absorbed, there is ample opportunity for dissipation of this heat. Between these two extremes there may be a point at which large amounts of heat are absorbed without adequate opportunity for dissipation and, as a result, extreme oxidation takes place

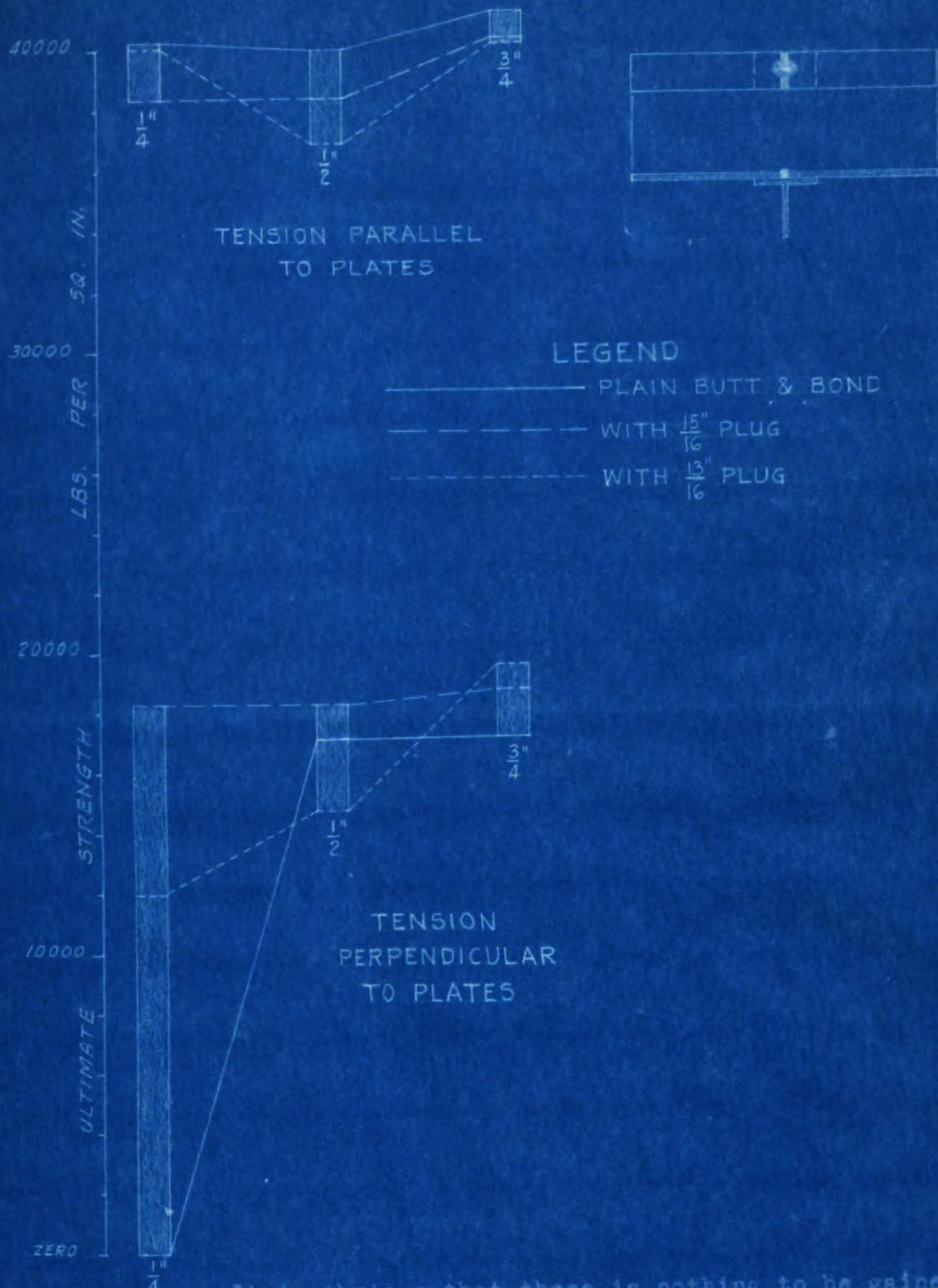
One thing not shown by this plate is that there was littl or no bond between the plates and the beam flange where the gap was only 1/4". The beam fell off during handling but the strength of the weld compared favorably with the other specimens

BUTT & BOND WELDS



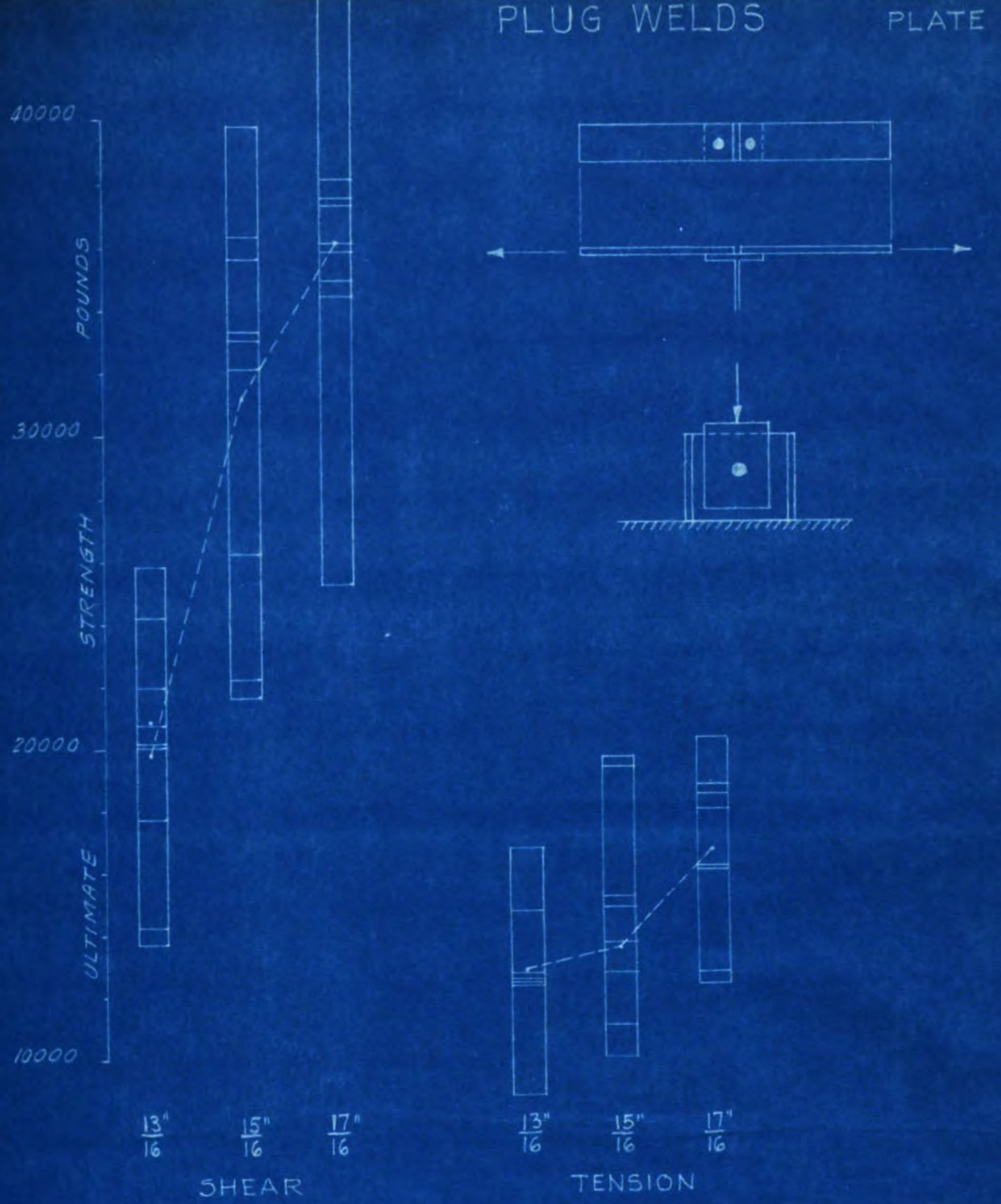
This plate shows the effect of the gap between plates upon the strength of a simple butt and bond weld stressed in tension perpendicular to the plates. The specimens made with only a 1/4" gap between plates failed in handling as did also some made with a 3/8" gap. It is obvious that too narrow a gap results in a joint with little or no strength under this kind of stress. It would seem that a gap 1/8" wider than the thickness of the thicker plate is the most efficient and economical. The strength of joints made with a 5/8" gap fell off in the same way as for welds stressed by tension parallel to the plates.

COMBINATION WELDS



It is obvious that there is nothing to be gained by the addition of a plug weld within a butt and bond weld which is to be stressed in tension parallel to the plates. Where the stress is to be tension perpendicular to the plates the addition of a plug weld increases the strength of the joint greatly. However preparation of plates for this type of weld is too expensive for ordinary usage

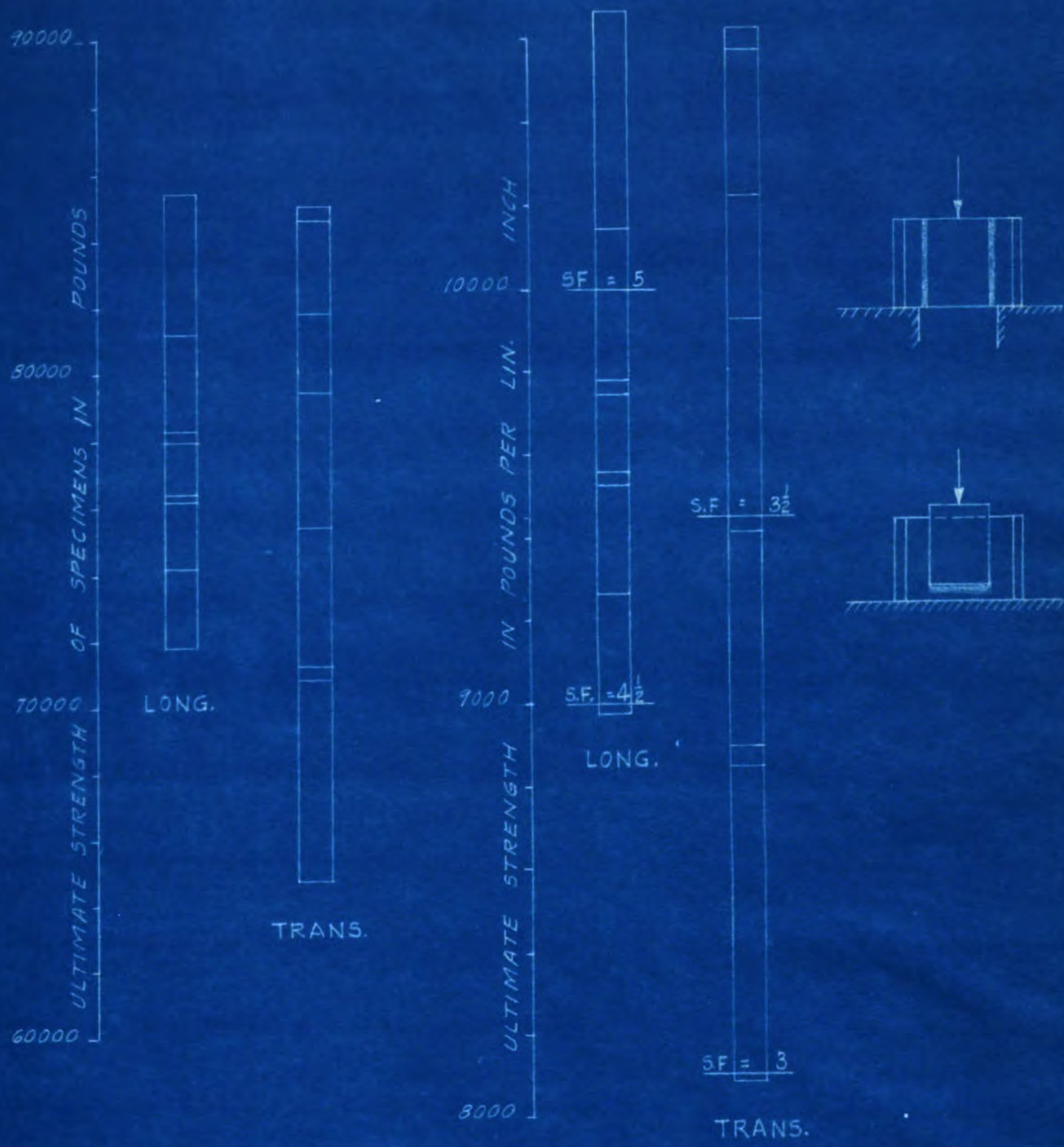
PLUG WELDS



The strength of plug welds is surprisingly high when on considers the small amount of weld metal to be placed Where the stress is applied as a compressive force acting in the plane of the plates it is evident that even a 13/16" plug weld can be assigned a far greater allowable stress than is standard for a 3/4" rivet in single shear. Where the stress is applied as a tensile stress in the plane of the plates the strength of the weld is much lower and this type of joint should be avoided.

STRENGTH OF FILLET WELDS

PLATE E



The data above is for 3/8" fillet welds connecting material of the same thickness. This type of weld is very strong and dependable regardless of how the stress is applied and should be used in preference to any other type of weld.

The safety factors shown are based on allowable stresses of 2000 lbs./lin.inch in longitudinal shear and 2700 lbs./lin.inch in transverse shear. It would appear that a uniform allowable stress of 2500 lbs./per lin.inch would be entirely safe for welds of this size.

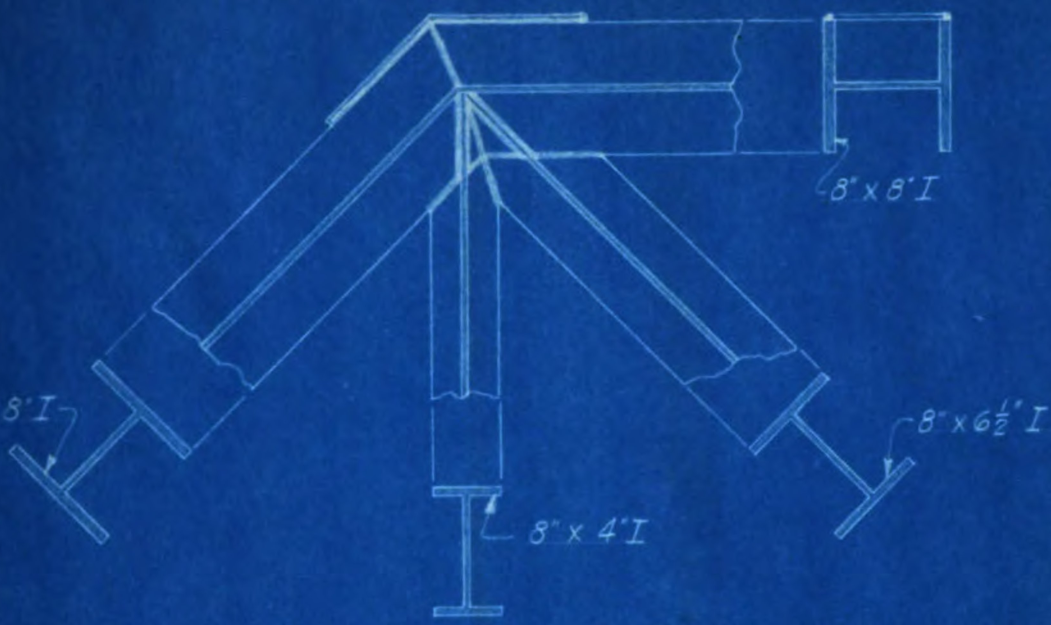
to stand the strain of bending to a very acute angle. In welded structures, as has been said before, all that is necessary for the making of a joint is for the members to overlap or intersect at an angle, preferably of more than 45 degrees, and no connection plate is needed. The only reason for preferring an angle of intersection greater than 45 degrees is that it is difficult for a welder to reach into deep and narrow V-shaped opening between members.

Plates II and III show in detail how certain of the joints in a plate girder or truss structure may be made. The arrangement of members shown is typical, but by no means the only way in which the joints may be made. Each structure will present it's own peculiar problems to be solved by the detailer and he will have to be guided by his own experience and ingenuity in their solution.

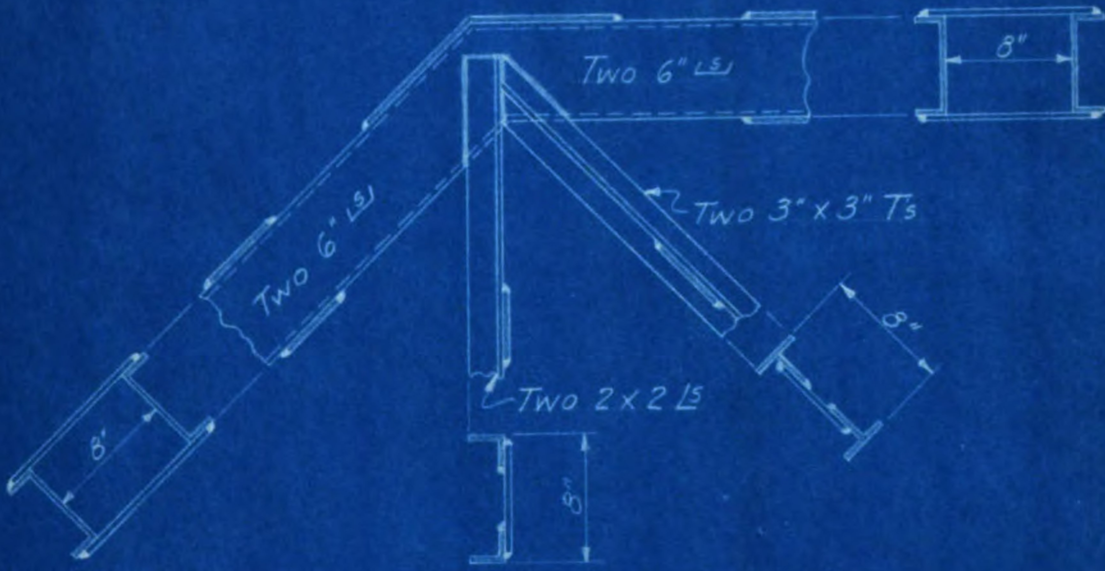
It will be noted that the joint between stringers and floor beams lends itself very easily to continuity. Under ordinary conditions, butt welding the top flanges at their intersection will suffice, but any doubt as to the efficacy of such construction can be relieved by the addition of a splice plate as shown. Also note that a fairly heavy seat angle is used both to carry shear and also to aid in erection.

The connection of floor beams to plate girders is a simple problem and the solution is self-evident. Note that the knee brace shown consists of only two plates and that one plate serves both as a part of the brace and as a stiffener plate for the web of the girder. The connection of floor beams to a truss is more difficult because of the lack of perimeter in contact. However, the choice of truss members with wide flanges and their arrangement in such a way as to provide flat planes will increase the amount of welding possible and in most cases there is ample opportunity to more than exceed the amount necessary according to the shear at the joint. There will be cases in which a gusset plate may be advisable and, while gusset plates are not contemplated in welded structures, there is certainly no reason why they should be considered as "out entirely."

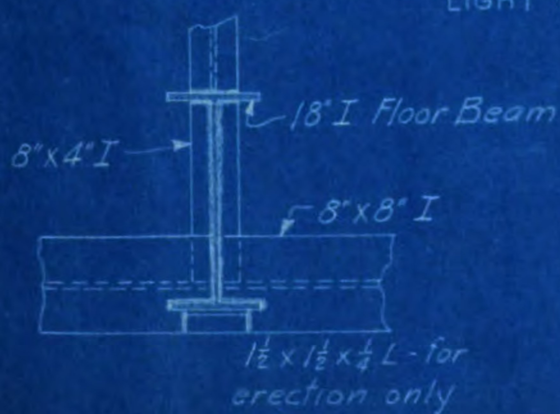
For heavy truss construction the wide flanged beam sections with the flanges square instead of tapered are the most suitable. There are many sections of the same depth, but differing in weight and it is possible to choose these in such a way as to make a very neat and attractive structure and at the same time maintain a maximum of strength. The joints are made by coping both flanges of one member so that it will fit within the flanges of the other, thus eliminating as far as possible, secondary stress due to eccentric joints and cross bending. Plate II shows a typical joint of a heavy truss in which the top and bottom chords are made of 8 by 8 inch column sections and the web members of lighter 8 inch beam sections. If additional chord section is required a plate welded to the upper edges of the beam flanges will provide added strength without adding greatly to the cost.

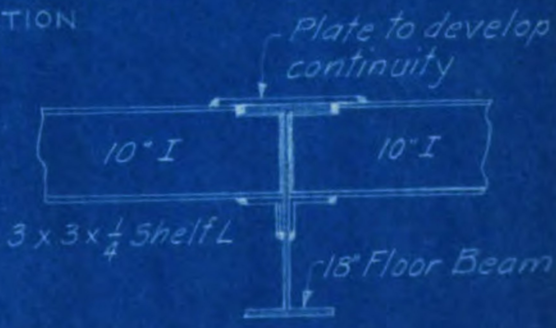


TYPICAL TRUSS JOINT
HEAVY CONSTRUCTION

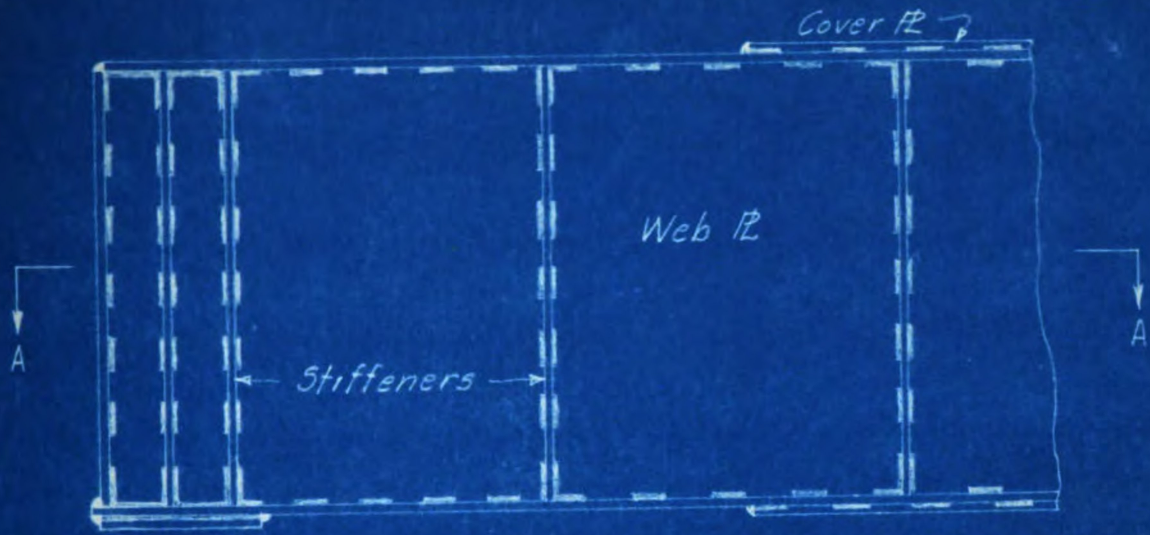


TYPICAL TRUSS JOINT
LIGHT CONSTRUCTION

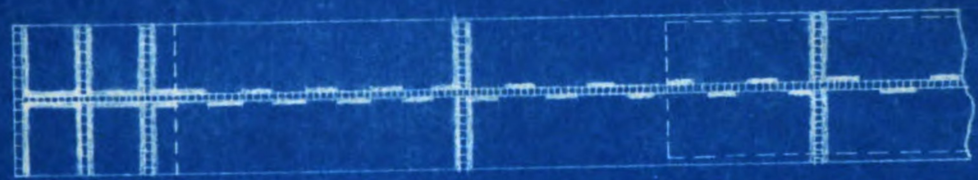




STRINGER CONNECTION

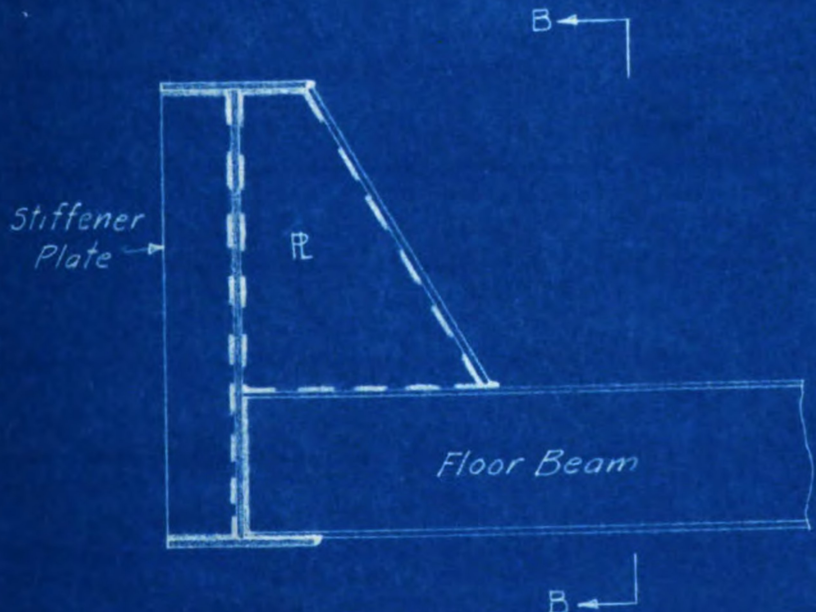


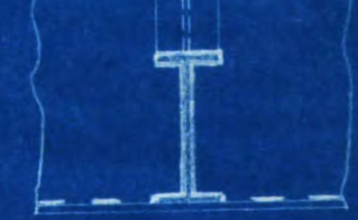
ELEVATION



SECTION A-A

TYPICAL PLATE GIRDER





SECTION B-B

TYPICAL BEAM CONNECTION & KNEE BRACE

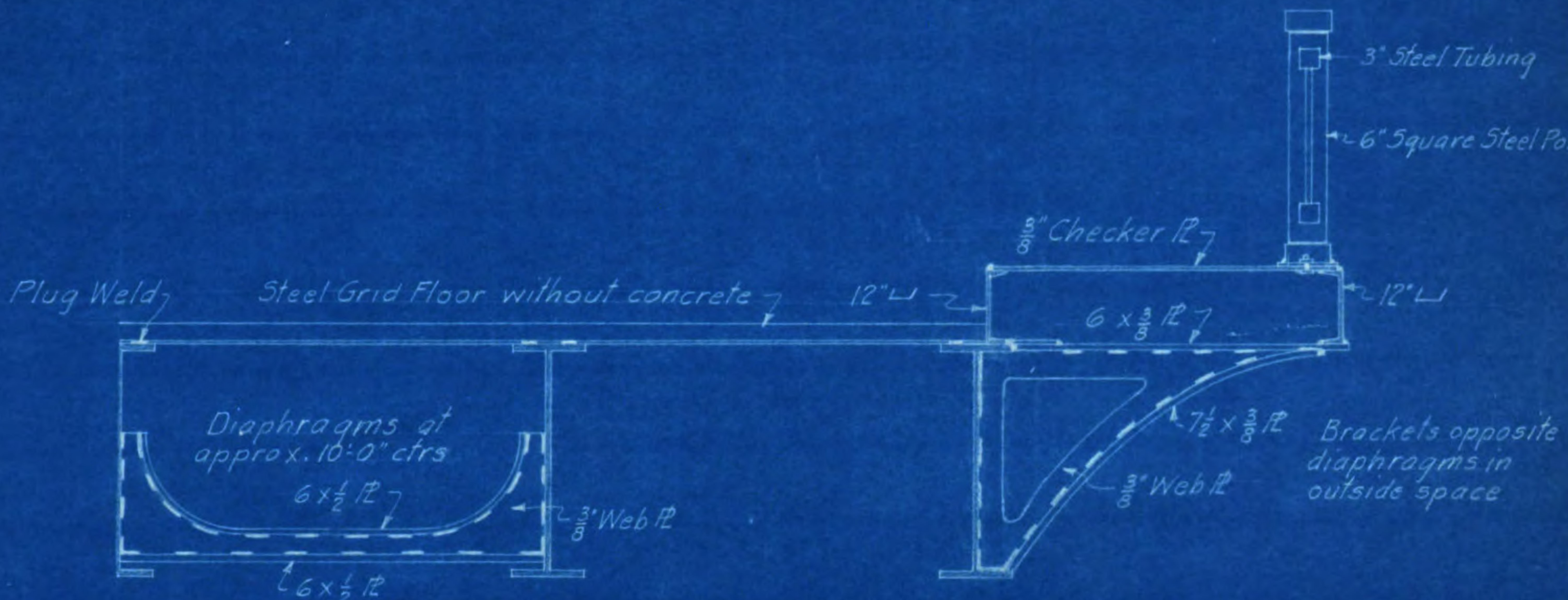
Lighter trusses may have chord sections composed of two channels back to back with either bar lacing or batten plates, and web members composed of light beams or tee sections or angles. The required distance back to back of the chord channels is governed primarily by the radius of gyration required for the compression chord but there should be no difficulty in adjusting this dimension to fit the depth of beam sections available. In this type of construction, joints would be made by inserting the web members between the channels.

In both cases, gusset plates would be almost completely eliminated. Splices in chord sections would be made by butt welding with or without splice plates. For trusses with inclined end posts, the haunch joint would be treated in the same way as a chord splice. The joint between the top and bottom chords at the ends of the truss is usually made by running the bottom chord through and fitting the top chord into it. The two members are welded together at all points of contact and splice plates welded to the outside of the members to further stiffen the joint. The bearing plate is welded to the bottom chord using stiffener plates if necessary.

Cantilever brackets are made in much the same way as are knee braces using a plate cut to the proper shape as a basis. Attachment of such brackets is a very simple matter for there are no reasons for avoiding welds in tension. If the tension at the upper edge is high, it is usually possible to add a tension member in such a way as to place the welds in shear.

So far this thesis has been essentially a discussion of the application of welding to bridge structures in general. The theory of welding and the apparata necessary has been discussed in detail. The actual making of a weld has been discussed and the difficulties encountered explained with the remedy for each such difficulty described. The strength of welds of different types has been touched upon lightly. The choice of members most suitable for welding has been outlined and typical construction illustrated. All of this information has been given to the end that the reader might have a clearer understanding of the problems involved in the design and erection of the all steel, all welded structure to be described in the balance of this paper.

Modern highway practice has established beyond reasonable doubt that the steel deck girder type of superstructure is the simplest and most adaptable for highway spans up to perhaps 90 feet in length and in relatively level surroundings. This type consists of steel girders resting on abutments and/or piers supporting some kind of floor slab. Diaphragms transverse to the center line of the roadway at about fifteen foot spacing are used to provide proper stiffening for the compression flange and to aid in distributing the loads. The slab may be



STEEL DECK GIRDER SUPERSTRUCTURE
DESIGNED FOR WELDING

either concrete or pre-fabricated steel. If concrete is used, it is customary to place one girder under the outside edge of the walk, but where steel is used, the sidewalks may be bracketed out from the outside roadway girder. It is quite customary to make the sidewalks easily removable so that the roadway may be widened without replacement of the original slab. For both concrete slab or prefabricated steel slab construction it has been found that a girder spacing of approximately five feet is the most economical and easiest to erect.

The advantage of this type of superstructure over concrete girder or arch construction and steel truss structure are easily recognizable. For highway loadings the dead weight of a truss and floor beam structure is entirely too great in relation to the live loads to be carried. Concrete structures require complicated falsework across the entire span and, of course, form work.

The steel deck girder span requires no false work and very little form work, in fact none at all if a steel floor is used. If a concrete slab is used whatever forms are necessary may be supported from the bottom flanges of the girders. Since the steel companies have adopted the wide flange section, and particularly since the addition of the 33 and 36 inch girders, it is possible to build up to 60 foot span length without going to plate girder fabrication and for the longer spans plate girders are less costly than truss construction. A single girder will not weigh over 10 tons and can be easily handled with the very common gasoline engine powered erection cranes.

During this period of rapidly increasing highway traffic the ease with which a steel deck girder superstructure may be widened is a primary consideration. Widening a truss span requires the moving of at least one of the trusses and an entirely new floor structure. Widening a concrete arch or girder span requires the erection of falsework to carry forms, and in many cases there is no way of bonding the old work to the new. Widening a steel deck girder superstructure requires only the addition of one or more new girders and the extension of the floor construction, provided of course that the abutments and piers are adequate in width. Such widening may be accomplished without interrupting the flow of traffic which is most important on heavily traveled trunk highways.

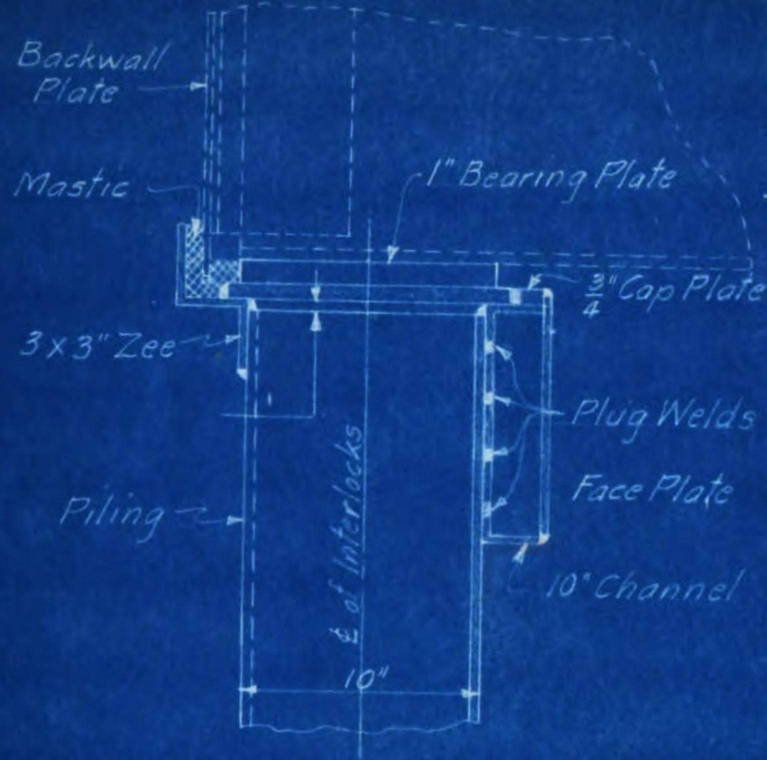
Another advantage which is of evident importance is the adaptability of this type of superstructure to skew construction. Skew trusses and monolithic concrete structures present major problems in design and erection. Skew construction for steel deck girder spans requires only that the ends of the girders be offset along the top of the supporting structures and the necessary changes in the floor slab. If it is desired to take advantage of the economies possible by the use of continuous beam design, butt welding the ends of the girders and the addition of a plate welded across the top flanges at the piers will amply provide for any negative moment developed.

Another important economical consideration is the extremely low dead weight. This factor is reflected not only in reduced cost of the superstructure, but also reduction in the cost of the substructure units. For a 60 foot span, the weight of a steel deck girder superstructure designed to carry R-15 highway loading is approximately 150 lbs. per sq. ft. of floor surface. A concrete T-beam floor for the same span and load will weigh more nearly 300 lbs. per sq. ft. of floor surface. For the deck girder span it is possible to cut the abutment cross section down to the minimum necessary for stability against over-turning without creating excessive soil pressures. This means less excavation.

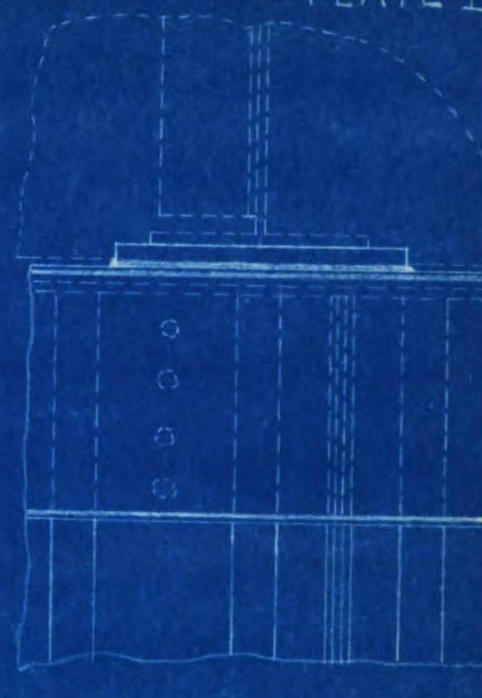
In comparing steel deck structures with short trusses it is necessary to take into account the addition of a pier which might not be necessary if truss construction were adopted. However, the cost of two 60 foot steel deck girder spans with a central pier is less than the cost of one 120 foot low truss due not only to the lower cost of fabrication and erection, but also to the reduction of the size of the abutments. It must be noted also that whereas the truss structure cannot be widened without replacing the entire floor structure and closing the bridge to traffic during such widening, the deck girder structure can be widened by the addition of girders and floor slab without interfering with the flow of traffic.

The all steel substructure units which the author would propose to use with the steel deck girder superstructure described above consist essentially of a row of deep arch web of steel sheet piling with a suitable cap structure. The piling forms a wall approximately 10" out to out to restrain the fill which, if properly designed and driven into stable soils, will also be adequate to carry the vertical loads imposed by the superstructure. Wingwall corners and any special sections required for skew construction may be fabricated from part sections of steel piles and such other structural shapes as may be suitable. It should be noted that welding does not work out satisfactorily for the fabrication of these special sections because of the extreme length of weld required. The effect of the heat absorbed is to release any internal stresses which are left in the metal by the rolling process and it becomes virtually impossible to produce a straight piece of piling without bend or twist.

For the abutment section, the cap structure would consist of a channel, a Z bar, and two plates, arranged as shown on Plate IV. The channel and plate on the stream side of the piling provides a certain amount of beam action at the top of the piles and tie the top of the piles together. Also these members form a coping which breaks the monotony of the row of piles and gives a sense of solidity to an otherwise light appearing structure. The

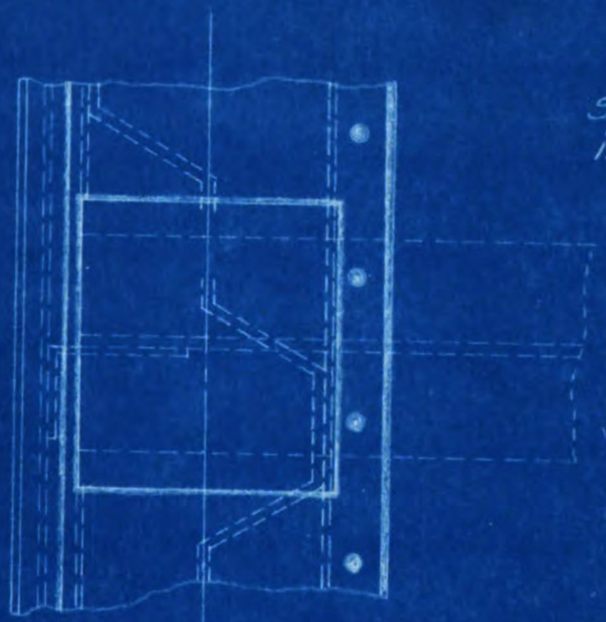


SECTION



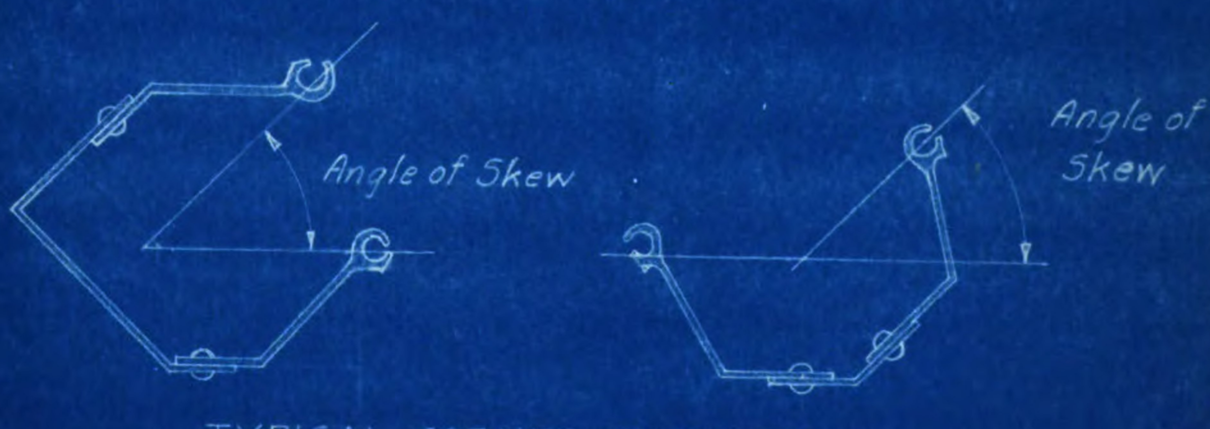
FRONT ELEVATION

Superstructure is shown in dashed lines



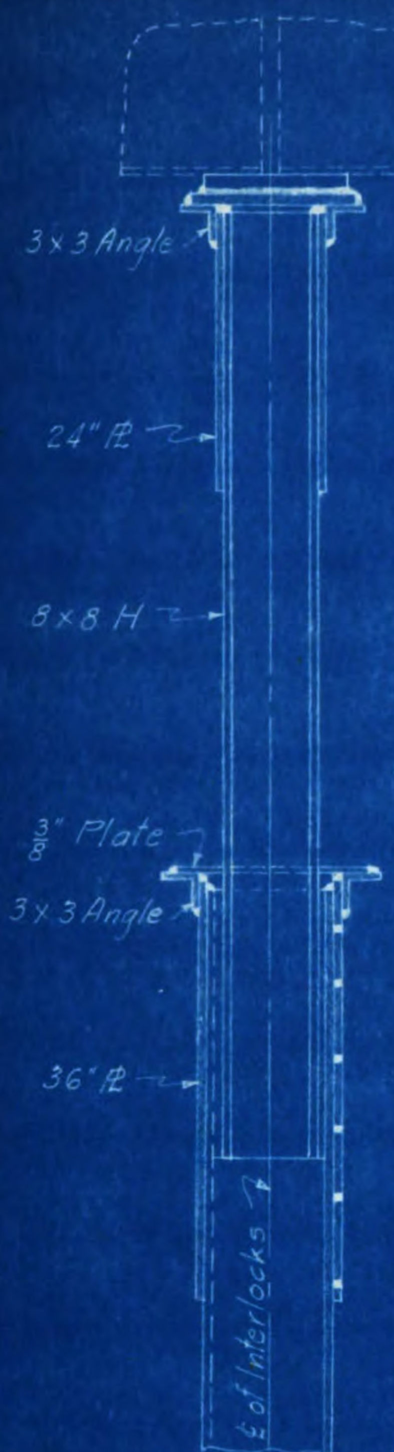
PLAN

DETAIL
OF
ABUTMENT

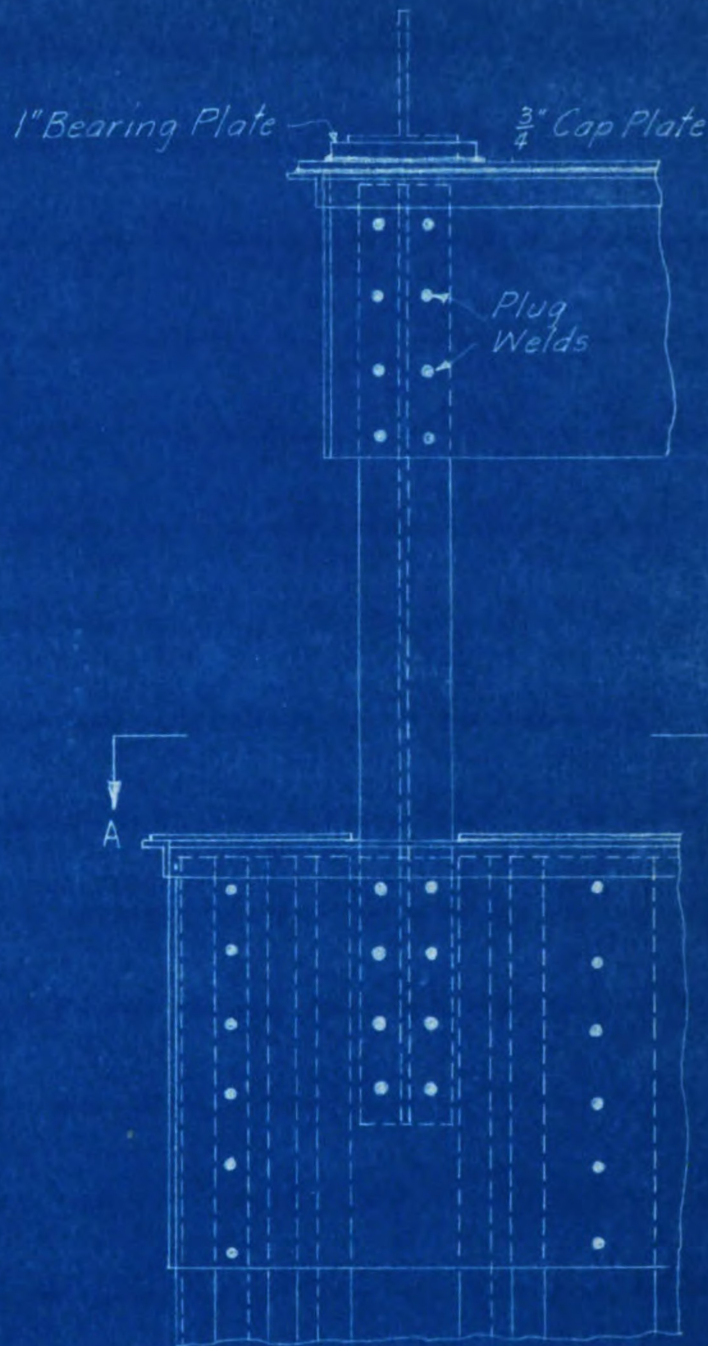


TYPICAL CORNER SECTIONS
FOR

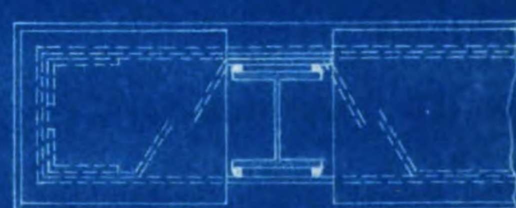
PLATE V



TYPICAL SECTION



ELEVATION



SECTION A-A

DETAIL
OF
END PILE SECTION

DETAIL
OF
PIER

Z bar at the back of the piling provides stiffness at the top of the piling and also forms a recess into which the backwall plate of the superstructure fits thus forming a cut -off for the earth fill behind the abutment. The cap plate is welded to these other members and forms a solid top for the abutment to which the beam bearing plates may be welded. Note that the top flange of the channel and the top surface of the Z bar are set an inch above the cut-off line of the piling to provide opportunity to weld all contacting faces of the piling to the cap structure and thus further stiffen the entire unit.

The pier section is more complicated particularly where the stream banks are steep and proper grade on the roadway requires a high pier. Again, the foundation of the pier consists of a single row of deep arch web steel sheet piling. In this case, however, it serves as a support for a column and girder construction of such height as is necessary to place the beam bearing plates at the proper elevation. The tops of the piles are made rigid by an angle and plate construction both plug and fillet welded to the piling. This rigidity of connection is absolutely necessary for the plate forms one side of the column anchorage. The end of the column is inserted in the opening between piles and plug welded to the plate on one side and to the face of the piling on the other, using as many welds as are necessary to carry the column reaction, and filler plates where necessary. A cover plate is then added to prevent debris from collecting within the piling wall. The plate on the outside of the piles should be quite deep both to stiffen the piling and also to prevent stream born debris from catching on the faces of the piles.

At the top of the columns a beam section consisting of angles and plates is used to support the bearing plates for the superstructure girders. The deep side plates are plug welded to the column flanges and serve as the web plates of a three sided box girder. The angles, which may be shop welded to the plates, add to the girder section and also serve to widen the support for the cap plate. After these portions of the pier structure have been assembled and welded, the cap and bearing plates are added and the pier is complete.

Architecturally, the appearance of these substructure units is not unpleasing to the eye. The outer surfaces except for the abutment piling, are smooth and unbroken. If it is desired an apron plate may be plug welded to the face of the abutment piling below the channel to hide the piling. Varying the method of placing the fill will create entirely different appearences. It would be possible to add the apron plate mentioned and carry the fill up almost to the bottom of the channel and thereby hide the piling entirely with a sodded slope. This treatment would be particularly pleasing for bridges located within a municipality where the banks of the stream are landscaped. The appearance of the pier could be altered by adding channels under the top plates at both levels to form copings. Probably the most important detail in regards to the appearance of the pier is that the face plates on the piling should be of such depth that low water will not expose the piling.

The advantages of the steel substructure unit over its counter part of masonry are numerous and all tend towards lower cost of construction. Let us consider them in the order in which the bridge contractor pursues his work. First, comes the laying out of the job. For masonry construction, the outline of the sub-footing or cofferdam must be staked out in its relation to the center line of roadway and the stationing thereof. This is a relatively simple operation if the angle of crossing is 90°, but for skew bridges is often a difficult and tedious job. After excavation has been completed and the sub-footing poured it is again necessary to get out the transit and lay out the outline of the footing. Laying out pier cofferdams and footings is an ever greater task if the stream is deep or swift moving.

In contrast to this, the layout for an all steel substructure unit requires only the establishment of a single line for the center of the piling. The instrument man sets up at the proper station and turns off the angle of skew. Two measurements along this line establish the location of the abutment corners, and the job is done. The ends of the pier piling can be established by accurately lining up the pile driver leads and following the driving of the first pieces of sheet piling to assure that they remain plumb.

Next comes excavation. For masonry construction, a large volume of earth must be removed from the cofferdams to obtain proper depth of footing and assure that the footings rest on a stable soil. Sheet piling cofferdams are usually necessary for pier excavations and very often for abutment excavations. Excavation is further complicated by the necessity for bracing the cofferdam walls and pumping out the water that leaks into the hole. In rainy weather conditions within the cofferdam are often so bad that the work has to be held up until the rain ceases. If the high flood water elevation is misjudged, pier cofferdams may collapse due to excessive outside pressure or be flooded. Extensive excavation during freezing weather is difficult and, normally, construction is postponed until reasonably warm weather arrives.

The excavation for a steel abutment takes one of two forms depending upon the topography of the site. If the ground is level a trench four feet wide and deep enough to permit the top of the piling to be driven to correct elevation will suffice. If the banks of the stream are steep, a shelf cut into the bank at the proper elevation is required. If the roadway approaches are on a fill section, it may not be necessary to make any excavation. The excavation for a pier consists only of removing logs, boulders, and other debris from the site of the pier.

The construction of a masonry foundation unit is, of course, essentially a field problem and very little work can be done except at the site. Large amounts of lumber, aggregates, cement, and other materials must be transported to the site and stored until needed and in a great number of cases storage of aggregates is a major problem to be solved. Erection equipment usually consists of a derrick, a concrete mixer, wheel barrows or trucks for the handling of aggregates and mixed concrete or perhaps a tower and chute system, and perhaps a power shovel, if the job is large. Concrete forms have to be built and properly braced and reinforcing steel wired into place. After concrete is poured there is a delay of one to three days before the forms may be stripped for possible use on another similar unit.

In contrast to this, the only field equipment necessary for the construction of a steel unit is a derrick with pile driving leads and a welding machine. The raw material to be used is steel which is not subject to damage by rain or high water. Special piling sections are fabricated in the shop and arrive at the site ready to drive. Other parts are made up according to the plans and are ready, when delivered at the site, to take their place in the finished structure. As soon as the piling for a unit is driven, the welders start the attachment of the cap structure and trim, and in a few days the pier or abutment is complete and ready for the superstructure.

Another great advantage is the ease with which this type of structure can be salvaged in case of need. Once in place a masonry foundation unit with the possible exception of one constructed of rubble masonry, is there to stay and there is no way of salvaging any of the materials therein for use elsewhere. The same is true of superstructure of masonry.

The steel deck girder superstructure with pre-fabricated steel floor is almost 100% salvageable. Railing and sidewalk are easily removable in sections. Also the floor. The girders may also be removed as soon as the diaphragms are cut loose. True, it is necessary to burn out the welds, but a good man with a cutting torch will cut the joints quickly and neatly with a minimum of damage to the parent metal.

The upper part of the pier and abutment units is similarly removed. Perhaps it may be necessary to scrap the whole of the abutment cap construction and cut the piling off. Nevertheless, the piling is only shortened slightly and may either be used in another bridge or sold for use in cofferdams. Research by steel producers indicates that copper bearing steel has a life expectancy of forty years so the first usage is not necessarily the last.

Most naturally there are some very definite limitations to be placed upon the use of the steel substructure unit. This type of substructure is not adaptable to use with heavy superstructures or for live loads in excess of highway loadings. After all, the load bearing capacity of steel piling is due almost entirely to skin friction with the soil and direct bearing on the end of the pile is of secondary importance. Therefore it is necessary that the imposed vertical loads be light and that the soil conditions be favorable to the development of skin friction to the greatest extent possible.

A complete and careful study of soil conditions at the site should always be made. Test borings should be made and samples of the various strata obtained so that the log of borings will present a complete and accurate picture of the soils through which the piling will pass. It is convenient to plot this information as a vertical section showing the elevations at which the various types of soil are encountered and their nature. From such a chart it is easy for the designer to determine how deep the piling should extend and whether sufficient lateral will stability be developed.

This matter of lateral stability is of utmost importance for the load on the piling itself is largely due to earth pressure, and therefore ability to withstand direct vertical loads is only one of the requirements for a stable structure. The top of the piling wall acts as a cantilever beam and is so designed. The length of the loaded section of the beam depends upon the depth of the soft soils encountered. Whether the beam will carry its load depends on the beam itself and also on the amount of restraint at the fixed end. Therefore it is necessary that the end of the piling penetrate well into a stable soil and for all practical purposes the length of the piling so restrained should exceed the length in unstable soil. It is probable that an upper stratum of coarse material such as gravel and a lower stratum of clay presents the ideal condition for both lateral stability and load bearing capacity. However, gravel alone makes a suitable foundation and the stiffer clay loams are also suitable. Clay of course will develop a great amount of lateral stability, but driving piling into stiff clay is a difficult process. The important point is that simply driving the piling down to a hard stratum is not sufficient.

Tie backs of one kind or another are always usable and in many cases are necessary. One of the simplest types consists of a few sections driven on a line at right angles to the piling wall and attached thereto by a T section of piling. Several of these may be used in a long wall. Another type consists of sections of piling driven down some distance back of the main wall and connected by rods jacked through the intervening soil so as to not disturb it. A well constructed "dead-man" will also serve. If such means will not develop adequate lateral stability, it will be best to abandon the idea of using the steel substructure and turn to a masonry structure.

The use of the steel substructure is also limited pretty much to the shorter spans because of the limitations of the steel deck girder superstructure. Such an abutment will not carry the heavy concentrations of a truss structure or of a long plate girder span. With girders spaced at about five foot centers, the reactions are small and there is ample opportunity for their distribution through out the wall. The cap structure shown will act as a beam of considerable strength and distribute the loads to the pile sections best able to carry them and prevent localized settlement from distorting the entire structure.

The difficulties encountered in the erection of the steel substructure unit are not many and their solutions simple. Probably the matters of length and alignment are the most common and troublesome. If the lengths of the members in the cap structure are based on plan dimensions it is necessary that the length of the driven wall vary but little from the dimension on the plans if excessive field alterations are to be avoided. Proper and adequate welding of the cap members to the piling also requires that the alignment of the wall be reasonably close. Where piling is used for wing walls, it is necessary that all returns from the superstructure fit as planned. Backwall clearances at the expansion end of the superstructure also depend on accurate alignment. And last, but far from least, the alighment of abutments and piers in multiple span bridges must be such that girders cut to length in the shop will fit.

At first glance the solution of the problems of length and alignment would seem to be to set up all the piling in its proper location before any of the pieces are driven. However, if the piling sections are 30 to 40 feet long, as is often the case, bracing such a wall becomes a very difficult and expensive process. The next thought is to drive the corner sections, taking all due precautions to locate them accurately and keep them plumb, and then fit the intermediate sections between. The difficulty encountered in this method is that of fitting the last section. If the wall is driven from both ends towards the center, it is very probable that the closing section would not fit and it would be impossible to make any adjustments in the line of driven piling. It has been the writers experience that the best solution of this problem is eternal vigilance on the part of the foreman and the inspector. Drive the center piece of piling first, keeping it in alignment and plumb. Then as each succeeding piece is driven, check the overall length of the wall.

If there is a tendency for the sections to crowd together, thus shortening the wall, put short pieces of $\frac{1}{4}$ inch bare welding rod into the interlocks as the pile is driven, and thus keep the joint as open as possible. If the wall tends to lengthen, use a clamp composed of a piece of $1\frac{1}{2}$" cold rooled round bar bent into a U shape with the ends threaded and a piece of 1 x 3 inch flat drilled to fit over the open ends of the bar. The bar can be put through the handling hole in the adjoining piece of piling or through a similar hole burned in the web of the piling and the position of the flat adjusted to keep a constant pressure on the piece being driven. By the use of such a device, it is possible to crowd the interlocks tight together and thus shorten the length of wall. Constant checking of the length of wall will easily determine whether the sections are going down according to the plans and as much as $\frac{1}{4}$" variation can be obtained in the fit of each pair of interlocks.

Proper alignment of the face of the wall is purely a matter of bracing and walers. In all cases, the piling should be driven between one pair of walers and two pairs about five feet apart vertically is better practice. These walers must be thoroughly braced and should be composed of at least 10 x 10 timbers. Holes burned through the webs of the pile sections will permit bolting the timbers together and to the driven sections. Then, if an instrument is set up and the plumbness of the driving leads constantly checked, it is relatively easy to drive the piling to a true line. Lining up the piling before driving has proved impracticable, for the same reasons mentioned heretofore.

After all piling has been driven, there is still the problem of the fit of the cap members. In general, this problem is best solved by working from both ends towards the center, and fitting the center sections in the field. This, of course, necessitates that the center sections be ordered at least 3 inches longer than plan dimensions would indicate. A good torch operator can burn off any excess neatly and the small gap left can be filled with weld metal and then ground smooth.

The alignment of the sidewalks or safety walks can also create a problem if too much faith is placed upon the accuracy with which the girder sections are constructed. There is no good reason for the web plate of a plate girder being off center, but the web of rolled sections may be off as much as ¼" and still be within the allowable tolerances of rolling practice. Therefore in lining up the superstructure girders, it is best to use the web as the controlling point. Variations in flanges can be nullified by proper allowances in the fit of members. This explanation is made because the fitting and alignment of the curbs and sidewalks depends normally on the accuracy with which the sidewalk brackets are located.

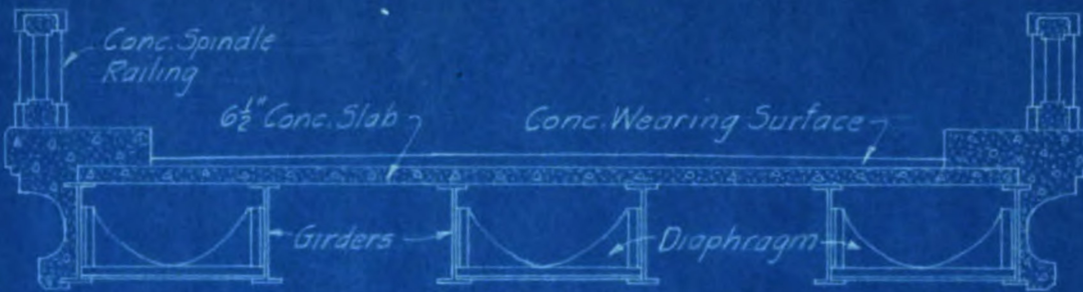
If the diaphragms are accurately built, the girder webs will be parallel and symmetrical about the center of the roadway. The sidewalk brackets are attached to the girder web and will also be in good alignment. Any variations in the relation of the curb line to the center of the top flange of the outside girder will then be unimportant. The author was called upon in one case where there had been careless workmanship in the fabrication of the diaphragms and the inspector had allowed this fact to be overlooked. An overwide diaphragm had been placed near the center of the span and the discrepancy overcome by deforming the girders to fit. As a result, the outside girder was decidedly rainbow shaped, and the sidewalk brackets correspondingly out of line. The amount of cutting and patching necessary to overcome this error took nearly two days to accomplish on a two span bridge and the final results were obviously the easy way out of a serious difficulty.

As will be noted on the drawing on Plate IV the backwall of this type of structure consists of a plate welded to connection plates which are in turn welded to the ends of the girder webs. The bottom edge of this backwall plate fits into the slot formed by the cap plate and the Z-bar of the abutment and it is of course important that there should be no failure of the expansion provision due to binding of parts. Since the alignment of the piling will be imperfect and since also it will be obviously impossible to make any adjustment of the piling any variations in alignment have to be corrected in the adjustment of the backwall plate. It is wise therefore to use a series of short plates about ten feet long and place them in their proper position before any of the connection plates are welded, either to the backwall plates or the girders. After the backwall is in place, the expansion slot should be filled with a flexible mastic which will prevent any dirt or stones from getting into the joint.

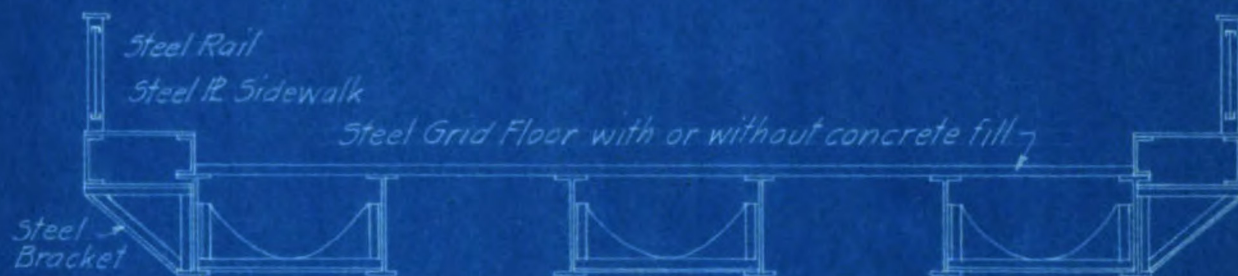
The method employed to attach the sidewalk brackets will depend to a great extent upon the possibility of future widening of the structure. If the roadway and sidewalks as planned are likely to be adequate for many years, it is best to weld the brackets in place and take advantage of the strength of weld metal in tension. If there is likely to be a need for widening, it is better to bolt the brackets and so design the location of holes that they can be used in the fastening of future diaphragms. In case the attachment of diaphragms requires more holes than are used in the attachment of the sidewalk brackets, the extra holes may be temporarily filled and the appearance of the structure improved by the use of bolts similar to those used to attach the bracket. This will avoid the burning of any holes necessary but not punched when the girders were fabricated. It is not recommended that rivets be used for they are difficult to remove and not satisfactory for the transmission of the tension at the top of the bracket.

There is also a likelihood that the railings may not line up well although careful fabrication should reduce this difficulty to a minimum. The attachment of the railing posts to the brackets should be by bolts to facilitate their removal in case of widening or accidental damage. Likewise, the panels of railing should be bolted into place. Any variations in fit can be easily overcome by slotting the holes used for attachment.

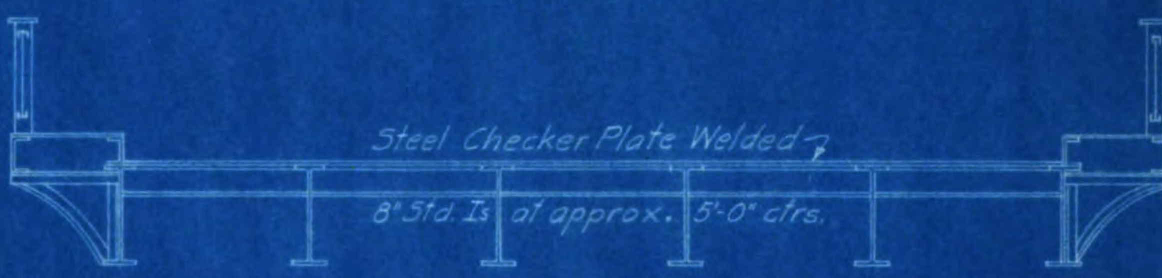
So much for the details of the fabrication and erection of the all steel, all welded bridge. The drawings which follow immediately are included to indicate definitely the types of structures with which the cost of the all steel bridge will be compared and represent in general the type of structures being built by the Michigan State Highway Department in the years 1929 to 1931. The comparison is based on the cost per square foot of roadway which seems the best unit to use. In as much as there are few of the all steel bridges in existence, it is impossible to obtain accurate figures on their cost based on a large number of jobs. The structures selected, however, represent typical construction, one an ordinary stream crossing and the other a highway overpass at a two track grade separation. The bridge was erected over the Looking Glass River three miles north of the Lansing City Airport and is of two 55 foot spans with 22 foot clear roadway and 2 foot 6 inch safety walks. The grade separation is at the intersection of State Trunk Line M-114 and the Grand Trunk Western Railway $2\frac{1}{4}$ miles east of Grand Rapids and consists of two spans of $26'-10\frac{3}{4}"$ and one of $48'-8\frac{1}{2}"$ with 42 foot clear roadway and 2 foot 6 inch safety walks. The author was placed in supervisory charge of the erection of the bridge and is therefore familiar with the problems to be solved in this type of construction and this job in particular.



STEEL DECK GIRDER
SUPERSTRUCTURE
CONCRETE SLAB FLOOR



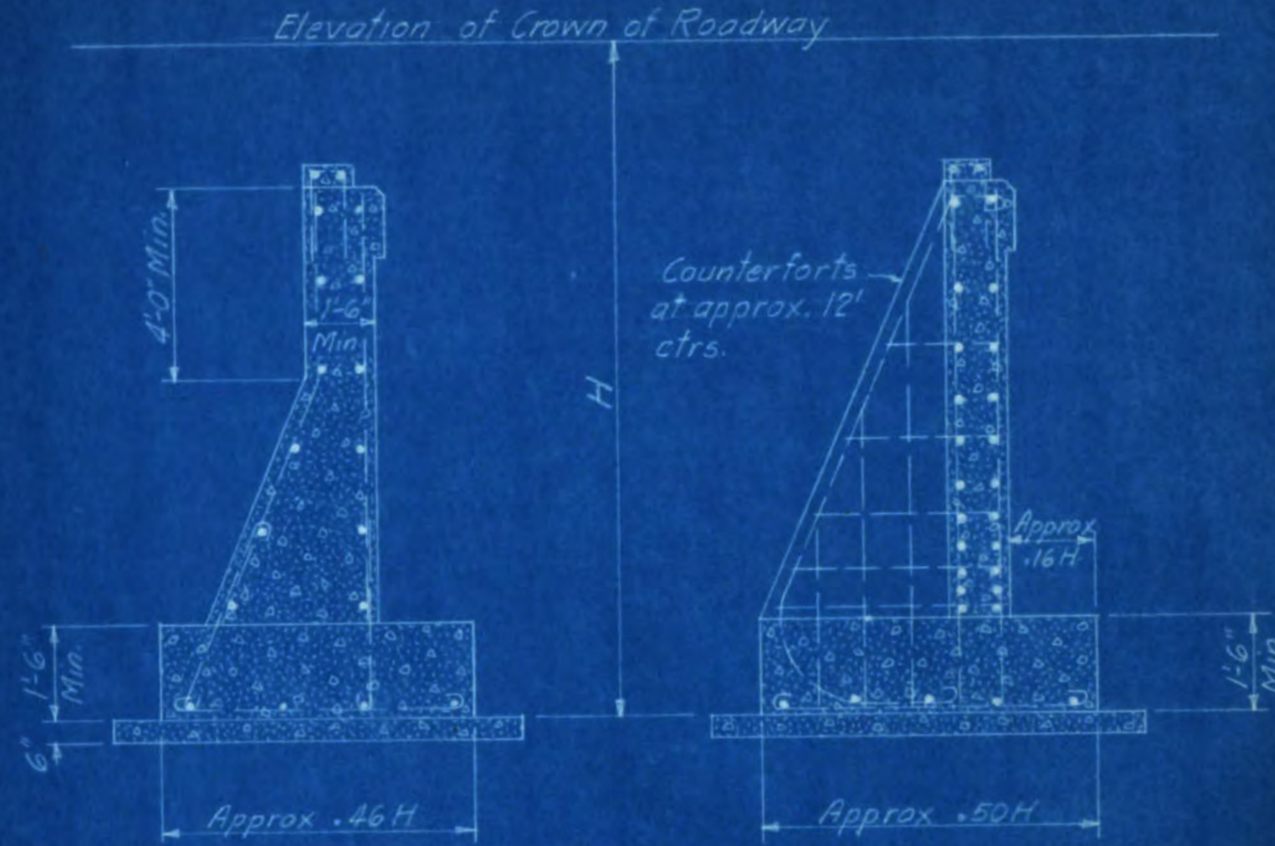
STEEL DECK GIRDER
SUPERSTRUCTURE
STEEL GRID FLOOR



STEEL DECK GIRDER
SUPERSTRUCTURE
BATTLE-DECK FLOOR

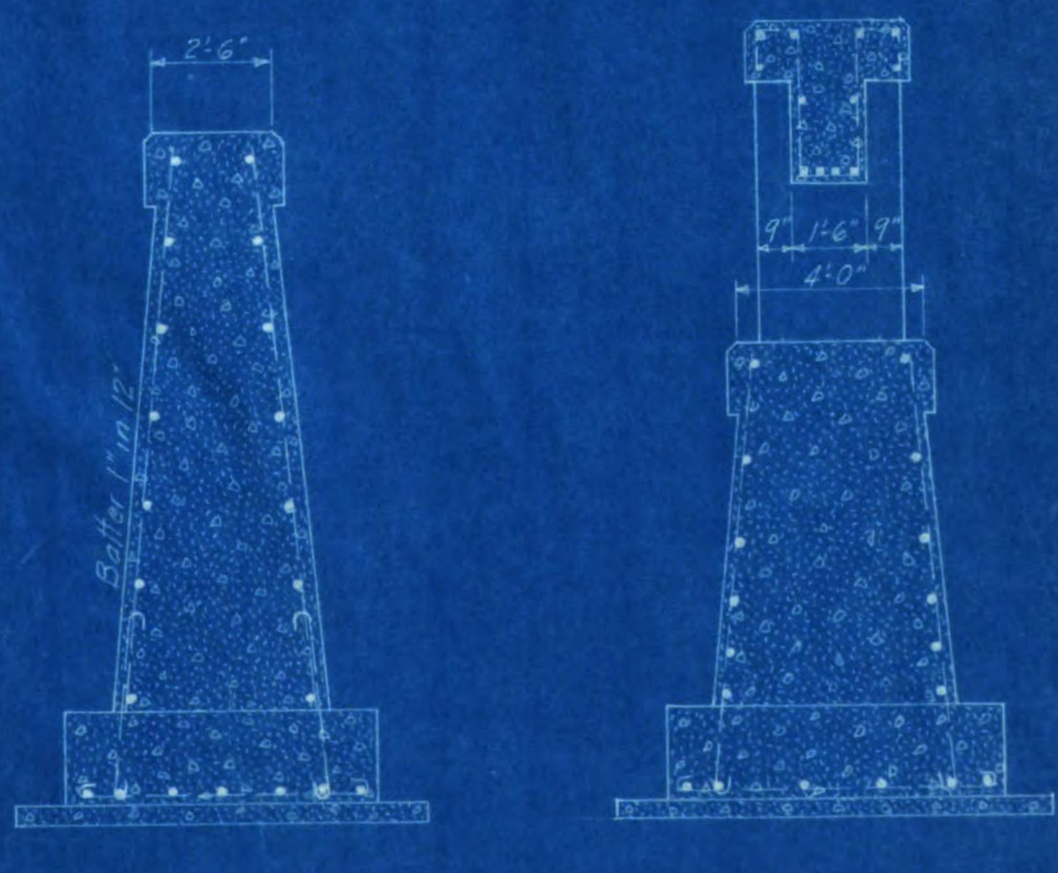
CONCRETE DECK GIRDER
SUPERSTRUCTURE

TYPICAL CONSTRUCTION
MICHIGAN STATE HIGHWAY DEPT.



SEMI-GRAVITY COUNTERFORT

TYPICAL ABUTMENT SECTIONS



GRAVITY COLUMN & GIRDER

TYPICAL PIER SECTIONS

The total cost of the bridge as reported in the Fourteenth Biennial Report of the Michigan State Highway Department was $16473.45 for 4220 square feet of bridge floor or $3.90 per square foot. The cost per square foot of ten other structures of the steel deck girder type erected on masonry substructure units during the same year and representing nearly 33000 square feet of floor area averaged $9.50 or more than double the cost of the all-steel job. A low steel truss with a floor area of 3232 square feet cost $8.80 per square foot. A reinforced concrete T-beam structure with a floor area of 4580 square feet cost $10.77 per square foot. These figures indicate very forcibly the great decrease in cost of construction possible by the use of this type.

The total cost of the all steel grade separation as reported in the same volume was $22000 for 4284 square feet of bridge floor or $5.14 per square foot. Two other grade separations of the steel deck girder type on masonry foundations, a total floor area of 2370 square feet, cost $19.00 per square foot. Two grade separations of the half through steel plate girder type on masonry foundations, a total floor area of 5830 square feet, cost $19.82 per square foot. It is interesting to note that, since the cost of a steel deck girder superstructure would not vary greatly from one job to another, the difference in cost between the various types of structure represents the saving possible by the use of the steel substructure construction.

Another method of comparison. The total cost of the 33000 square feet of floor area of the steel deck girder spans on masonry substructures was $311,571.68. Using the unit price figure computed above for the same type of superstructure on steel substructure units, this total cost would be reduced to $128,700.00, a saving of $182,871.68. There is no doubt that some of the structures included in this group were not of such nature as to make the use of steel substructure unit advisable. Nevertheless, a saving of even a half of the $182,871.68 would be a major item and would represent at least two structures of the average type.

The one great and all important question which has arisen in regards the type of structure proposed in this thesis is that of usable life. Since the superstructure would be essentially the same regardless of what type of substructure units were used, it seems fair to disregard that phase of the problem. We are all used to thinking of masonry as a practically permanent type of construction, and that steel is subject to oxidation to the extent that a life of perhaps 20 years is all that can be reasonably expected without major replacements. However, modern improvements in the metallurgy of steel are changing this concept rapidly. The use of small amounts of copper as an alloy has practically doubled the expected life of steel exposed to the weather.

Alternate wetting and drying is now considered the most harmful type of exposure and this exposure is inevitable in a steel pier unit. We must not, however, forget the many instances of perhaps a different application of the same material which prove that with reasonable maintenance the life of a steel structure under these conditions is much over 20 years. The

Brooklyn Bridge in New York City was built over fifty years ago and, while it is now very inadequate to meet the demands placed upon it, the original steel cables are still in use. The huge bridges being built in the San Francisco area are certainly not expected to have a usable life of only 20 years for the probability is that they may not be paid for by that time. The Carnegie Steel Company in their handbook on the use of steel piling make an extensive report on the condition of piling removed at Pittsburg which had been in place for twenty years. This piling was a 12 inch, 35 lb. section, known at the time as M-101 of ordinary carbon steel with a carbon content of only .01 percent and was driven ten feet into hard mud and gravel with the tops extending 18 inches above normal water level. Sections were cut at the top of the pile, at the water line, at the middle of the pile, and at the bottom of the pile and the loss of metal due to corrosion and other causes carefully determined. At the water line there was a loss of 14.9 percent or less than one percent annually over the time the pile was in place. Because of mining activities in the Pittsburg region, the water of the Monongahela River contains more or less free sulphuric acid which during the low water months runs as high as 100 parts per million. This, therefore, was a severe test of the piling and proves definitely that even plain carbon steel has a usable life of more than twenty years. Copper bearing steel can therefore be expected to have a life of at least forty years or more and past experience with masonry has shown that in the length of time mentioned, the best of concrete structures will need very major repairs.

There is no questioning the fact that maintenance charges on the all steel bridge will be higher than for masonry. Not that there will be major items for the replacement of structural members, but that a steel structure has to be painted quite frequently if it is to endure. The cost of painting the average steel deck girder bridge on concrete abutments is shown by the report of the Michigan State Highway Department for the year 1932 to be approximately $250.00. Suppose for purposes of discussion we double this figure to take into account the additional surfaces to be painted, in the case of the all steel bridge, making the charge for painting $500.00. Taking the specific case of the bridge over the Looking Glass River north of the Lansing City Airport, it has been shown that the saving in first cost of this structure over an ordinary steel deck girder bridge on masonry abutments would be approximately $20,000. Certainly this amount would represent a large number of paintings, so many that it is safe to assume that the increased cost of maintaining the all steel structure will never become a major item in the choice of bridge type for a site where the all steel bridge can be used.

SUMMARY

It has been the purpose of this thesis to set forth in some detail, the possibility of fabricating and erecting an all steel bridge by the process of arc welding, and to prove that the type of bridge proposed is practical from both an engineering and a financial standpoint. To this end, the process of electric arc welding was discussed in considerable detail in order that the reader should have a clear understanding of the methods to be used and the results that could be reasonably expected. The data included in regards the strength of welds made with the use of bare wire electrodes shows that such welds will permit a factor of safety of four which is common for steel construction It must also be borne in mind, in this connection, that the use of bare wire electrodes has been almost discontinued in favor os the heavily coated, shielded arc electrode which will practically guarantee an increase of twenty-five percent in the strength of the weld over similar welds made with bare wire.

The type of structure proposed was then described in great detail with drawings added to show typical details of construction. The advantages and disadvantages of the new type were discussed and compared with masonry construction. Many of the details of fabrication and erection difficulties were set forth and the appropriate solution explained. The limitations to be placed upon the use of the new type were set forth. And, finally, the cost of building and maintaining this new type of bridge structure was compared with similar costs for masonry or riveted steel structures.

The author believes that the following conclusions may be arrived at and that there is ample justification for so doing.

A. That arc welding as a means of fabricating and erecting steel structures is a dependable process and that its use will permit many economies in design and erection.

B. That a bridge design based on the use of steel sheet piling for supporting abutments and piers is sound from an engineering standpoint.

C. That the limitations which have to be placed upon the use of such a structure and the difficulties to be expected in fabrication and erection do not preclude the general adoption of the proposed methods.

D. That the structure proposed can be built at a considerable saving of both time and money.

E. That the life of such a structure will compare favorably with the life of bridges constructed on masonry abutments and piers.

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