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METALLIC ARC WELDING AS APPLIED TO BRIDGES AND ALLIED STRUCTURES WITH SPECIAL REFERENCE TO THE NORTH WESTERN RAILWAY.

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Joining steel or iron members of a structure of a bridge together by Metallic Arc, or Electric Arc Welding as it is often called, instead of the more usual method using rivets or bolts is fast becoming a very important branch of Structural Engineering. In Australia and Europe great strides have been made in this Art and in the United States of America it is said to be the eighth industry in order of importance ; but in India, the practice of Metallic Arc Welding is only in its infancy and, before going on to describe some of the work done on the North Western Railway, it is thought that a short description of Electric Arc Welding will be of interest to Members.

Electric-Arc Welding.

The process of electric-arc welding is the fusion together of two surfaces of metal so that the junction shall have all the qualities of the parent metal. For this, an electrode is applied along the line of the weld and, by the heat of the electric-arc, is fused into the parent metal. Thus in welding, the operator actually controls a small steel furnace at the weld ; the process calling for accurate control from the metallurgical point of view.

The quality of a welded joint depends on three things : the electrode ; the temperature of the weld during fusion, which is in direct relation to the current used ; and the skill of the operator.

Electrodes.

Electrodes may be classified in three types, namely, bare wire, paste coated and asbestos covered electrodes.

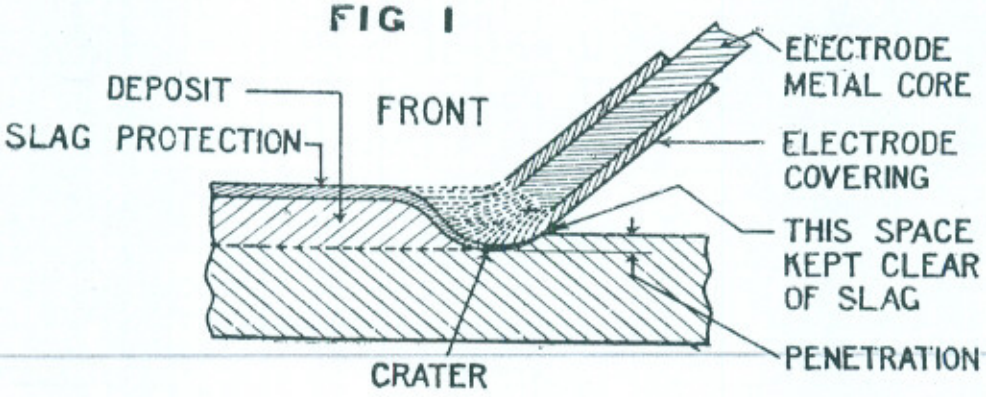
Both the oxygen and nitrogen of the air are extremely active chemically with white hot or molten iron combining to form oxides and nitrides, the principal causes of weld embrittlement. The longer the arc the greater the air contact with the hot passing metal and the higher will be the oxide and nitride content of the resulting weld deposit.

Welds made with bare wire electrodes are totally unprotected from the atmosphere during fusion ; and, as they require a higher current and longer arc than covered electrodes, the resulting deposit is both porous and brittle.

Paste coated electrodes are cheaper than the asbestos covered one but the slag produced has a comparatively high melting point and does not become sufficiently fluid to afford complete protection from the oxidizing influence of the atmosphere. Also the paste, chiefly chalk has a tendency to flake off as the electrode gets hot leaving the original bare wire.

When welding with asbestos covered electrodes, however, the process is likened to modern steel making where a flux, having a lower melting point and a lighter density than the steel, added to the charge, protects it from the atmosphere. Fig. 1 shows how the asbestos protects the weld in a similar manner.

FIG 1



So far as welding is applied to Bridge work where the ductile properties of the parent metal must be retained, the choice of electrodes used on the North Western Railway has been confined to those covered with asbestos. Quasi Arc Mild steel electrodes are used for the general welding of wrought iron or mild steel structural work and Quasi Arc Uranium electrodes, having a large manganese content are employed on work subjected to heavy stresses, particularly fatigue stresses, and on work where the adjacent metal has to be hot-worked.

The following table gives the percentage chemical content of the steel wire cores of these electrodes, the parent plates which are of basic open-hearth steel of 28—33 tons per square inch breaking strength rolled to British Standard Specification No. 15, and the deposits after welding :—

	Carbon.	Manganese.	Phosphorus.	Sulphur.
Mild Steel Core	0·110	0·500	0·034	..
Uranium Core	0·110	3·130	0·046	..
Structural Steel Plate ..	0·20	0·50	0·03	0·04
Weld Deposit (average) : Mild Steel Electrode	0·06	0·10	0·04	0·03
Uranium Electrode	0·06	0·52	0·04	0·03

In order to make the work as easy as possible for the operators, the different kinds of electrodes have been limited. In addition to the Quasi Arc electrodes, referred to, M. W. P. asbestos covered "Cresta," "Bluex" and Special "A" electrodes have also been tested and used on some of the works.

Strength of Weld Metal.

The strength of weld deposits from a number of different makes of electrodes have been investigated and the following table gives the results obtained from tensile tests on specimens of solid weld metal prepared from a succession of runs of No. 8 gauge electrodes in a wide angle butt weld. These deposits were 1, $\frac{1}{8}$ inch thick and seven inches long and were turned down to $\frac{3}{4}$ inch dia.: at the ends for gripping and to $\frac{3}{8}$ inch dia.: over a test length of just over 2 inches. The same operator and plant were used throughout and some four specimens were made from each class of electrodes.—

Type of Electrode.	Yield point Tons/sq. in.	Ultimate strength Tons/sq. in.	Percentage elongation on 2 inch length.
Quasi Arc Mild Steel Asbestos covered	20·1	25·6	10%
* Ditto Uranium Asbestos covered	19·6	28—32	25%
* Mild Steel Bare wire	..	20—26	6%
M. W. P. Cresta Asbestos covered ..	23·6	29—5	20%
M. W. P. Bluex Asbestos covered ..	19·9	24·0	7%
M.W.P. Special A Asbestos covered and Mild Steel Plate (B. S. S. No. 15) ..	23·8 15/20	28·2 28/33	9% on 8 inch 16/20%

* These figures, selected for comparison, have been taken from tests made in England.

But as tensile tests are not readily suitable for the comparison of weld deposits subject to severe impact stresses, four dozen standard Izod Impact Test Specimens were prepared with a butt weld at the point of rupture. These were made by the same operator, using No.10 gauge electrodes of four different classes and the results of the tests are as follows :—

Quasi Arc Mild Steel—

15 ; 25 ; 49 ; 14 ; 35 ; 26 ; 21 ; 20 ; 33 ; 33 ; 18 ; 15 foot lbs.
average .. 25 foot lbs.

M. W. P. Cresta—

46 ; 42 ; 42 ; 41 ; 32 ; 34 ; 28 ; 34 ; 54 ; 30 ; 38 ; 37 foot lbs.
average .. 38 foot lbs.

M. W. P. Bluex—

46 ; 24 ; 32 ; 37 ; 32 ; 35 ; 42 ; 40 ; 40 ; 20 ; 20 ; 38 foot lbs.
average .. 34 foot lbs.

M. W. P. Special A—

68 ; 42 ; 54 ; 62 ; 69 ; 58 ; 70 ; 51 ; 69 ; 82 ; 62 ; 60 feet lbs.
average .. 62 foot lbs.

The variation in these results shows the diversity in the strengtl of weld deposits which must be anticipated due to the human factor the operator who prepared these tests having had three years experienc and being a reliable and conscientious worker.

The average result obtained on Izod specimens of mild steel (B. S. S No. 15) gives a figure of 25 foot lbs., thus showing that most of the weld tested were of a tougher structure than the un-treated parent metal. Thi does not, however, refer to the parent metal immediately adjacent to th weld, which will reveal a coarse crystalline structure due to its havin been heated above its critical temperature.

In addition a number of $\frac{1}{2}$ inch butt weld specimens made by differ ent operators using Quasi Arc Mild Steel electrodes have been tested Two plates $9\frac{1}{2}$ inch long by $4\frac{3}{4}$ inch wide were joined by a butt welc along the $4\frac{3}{4}$ inch edges. They were then sawn down the middle anc one of the two 19 inch $\times$ $2\frac{1}{4}$ inch specimens, with the butt weld a right angles to line of pull, tested in the tensile testing machine. The other specimens were tested in an alternating stress testing machine installed in Jhelum, in which, clamped rigidly with the centre of the butt welds co-incident with the mouth of a vice on the machine, they were bent backward and forwards inducing an extreme fibre stress of $\pm$ 40 tons per square inch on both faces of the joint. The numbers of reversals before fracture occurred were noted and the results compared.

The following table gives the results of these tests :—

	No. of reversals.	Ultimate stress Tensile test.
Weld Deposit	1,260	22·7 tons per square inch.
	1,450	27·5 ,, ,,
	2,800	28·9 ,, ,,
	3,050	25·5 ,, ,,
	2,450	27·4 ,, ,,
	840	17·9 ,, ,,
	840	24·2 ,, ,,
	1,950	24·7 ,, ,,
Mild Steel Plate	24,000	28—33 ,, ,,

The apparent lack of continuity between the results is interesting and it shows the necessity for investigating the ductile qualities of weld metal subject to suddenly applied stresses. This investigation is being continued.

The accompanying photo-micrographs (Plate No. X) of the junction between a weld made with Quasi Arc Mild Steel Electrodes and Mild Steel Plate (B. S. S. No. 15) show the change over in the crystal structure of the metal at an arc-weld junction. The parent metal, mild steel plate, shows a normal crystal structure whereas the weld deposit reveals a very fine grained structure. The parent metal immediately adjacent to this weld, having been heated far above its critical temperature, reveals large crystals, which are an indication of a brittle metal. An interesting feature of these photographs is that there is no sharp line of demarcation between the weld and the parent metal such as is found in an ordinary forged weld.

The coarse crystal structure can be rectified by normalizing the metal, that is, by heating it above the upper critical point and allowing it to cool off undisturbed.

Current.

Either Direct or Alternating current may be used with asbestos covered electrodes which although generally attached to the positive pole, can be attached to either pole.

Voltage.

The current pressure required to weld with covered electrodes is about 30 volts on Direct Current and 70 volts on Alternating Current circuit; but there are instances when, owing to the resistance of the slag or other causes, a pressure of 100 volts may be required.

Amperage.

The amperage to be used depends upon the cross sectional area of the electrode, its covering and the thickness of the parent metal. The following table gives approximately the correct amperages for use with asbestos-covered electrodes used on structural steelwork:—

Electrode.	Current Range.
14 s.w.g.	35 to 45 amps.
12 ,,	65 to 75 ,,
10 ,,	80 to 95 ,,
8 ,,	105 to 120 ,,
6 ,,	125 to 140 ,,
4 ,,	150 to 170 ,,

The penetration of the weld metal with the parent metal is dependent on the temperature developed, which in turn is in direct relation with the current used. Too high a temperature in the metal adjacent to the weld will enlarge the crystal structure and render the joint brittle and liable to fracture under reversals of stress. But too low a temperature may result in a lack of penetration and a consequent weak joint.

Unreliable welders are apt to use an excessive amperage as it is easier to maintain the arc and the work is done more quickly. Nevertheless

the use of too high an amperage is a danger to be closely guarded again and, for this reason, the choice of the cheaper brands of paste coate electrodes, demanding a higher amperage is avoided.

In the Bridge Workshops of the North Western Railway two singl operator sets have been laid down. These generators are belt drive from the workshop shafting, and so long as the shafting is run at a fairl constant speed, very satisfactory results are obtained; but, any cor siderable slowing down of the shafting due to the operation of heav machinery off the same drive, is liable to cause a fluctuation of amperag and a consequently poor weld.

The original portable plant in use on Bridge Work in the field con sisted of a single-operator generator direct coupled to a four cylinde 11·9 h. p. Dorman petrol engine. The whole assembly was mounte on a frame and weighed about 16 cwts.

These units, however, have been superseded by the much mor economical crude oil engine.

The portable Welding Sets in the Bridge Department at the present time consist of five single-operator generators, all of which are driver by crude oil engines. Two are belt driven from 12 h. p. Petter oil engines, which have been adapted for this work by the department. These machines, whilst using slightly more fuel oil than the newer Diesel sets, have been in constant use for the past two years. The remaining two generators direct-coupled to single cylinder 11·9 h.p. Gardner oil engines were purchased complete about 18 months ago. Despite the economic fuel consumption of these high speed Diesel power units running at 1,450 R. P. M., the excessive vibration set up combined with other defects in the design have resulted in expensive piston replacements, and a comparatively short lease of life. For future development, it is intended to sacrifice extreme economy in fuel consumption for more lasting and reliable multi-cylinder Diesel power units running at a speed of 600 to 800 R. P. M.

The following comparative statement gives the relative costs of running these plants.

Plant.	Approximate consumption and cost of consumable stores per 9 hour day laying down 48 feet of single run of 10/5 weld.					
	Fuel. gal.	Rate per gal. Rs. a. p.	Lubricating oil.	Rate per gal. Rs. a. p.	Total cost. Rs.
Petrol ..	6·7	1 11 6	1 gal. Mobil oil.	2 4 0	13·78
Petter ..	5	0 4 0	½ gal. Mobil oil A	2 4 0	2·37
Gardiner Diesel	3·3	0 4 0	¼ gal. Mobil oil A	2 4 0	1·39

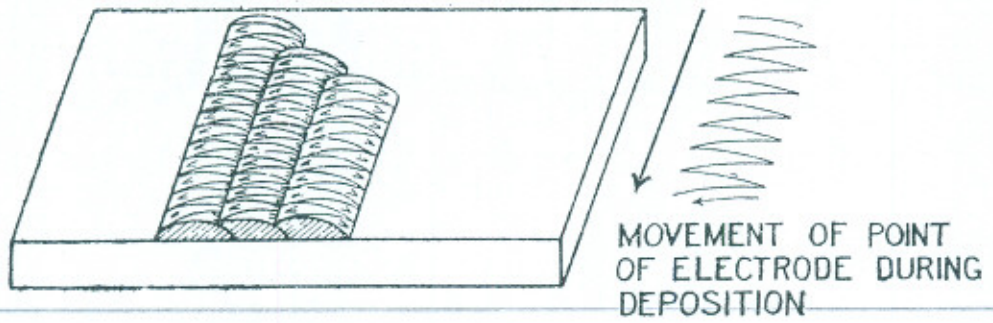
These figures do not include any allowance for maintenance.

Skill of the Operator.

Some idea of the importance of the operator's skill may be judged rom the comparatively high wages paid to men of five years and upwards xperience. The actual putting down of the weld must be left to the kill of the operator.

Looking through the screen during the deposition of the metal, the perator easily distinguishes the slag from the metal. The former appears brilliant red whilst the metal looks like a dull red jelly. By a motion hown in Fig. 2 the operator works the bright red slag from side to side f the weld ensuring that none is allowed to get underneath the metal.

FIG. 2



Before commencing a weld, it is essential that the metal to be joined hall be absolutely clean. All mill-scale, rust or paint is removed and ometimes the skin surface of the metal itself is chipped off. After each un of weld metal has been laid down in a multi-run joint, the slag is emoved by the use of a light chipping hammer and wire brush. If this s not done, the slag which remains, may be trapped beneath subsequent uns. When the operator finishes an electrode, he removes all slag from he crater, where he has left off before proceeding with a fresh electrode.

In vertical welding it is better to commence at the top and work own as this makes it easier for the operator to "flow" the slag down ver the metal keeping it outside the weld. Overhead welding is defi- itely more difficult than vertical or horizontal flat welding. The opera- or laying down a weld overhead is provided with a screen-helmet, gloves nd fireproof overalls. The electrode is held practically vertical below he weld with the point held close up to the work. If the point is held oo far away a large arc results and little or no metal is deposited. For verhead work smaller gauge electrodes are used and a larger number of uns specified.

There is no danger to the staff working on a welding job so long as hey do not look at the arc without using a proper screen consisting of pecially coloured glasses. If the eyes are directed to the naked arc, a emporary pain will ensue making the eyes feel as though they were full f dust. This complaint, known as arc eyes, is felt at its wo st after the lays work is over and the patient is going to sleep. But at its worst t is only temporary, going off in 24 hours.

Slight shocks are unavoidable under certain conditions, *e.g.*, in we weather, but these are rarely dangerous. Alternating Current gives th more severe shock, and for this reason, Direct Current is often specifie when workmen are welding on high staging.

Testing Welded work.

From time to time it is usual to make test specimens which, i possible, are cut from the work. If this cannot be done, the operato prepares specimens in exactly the same position, *i.e.*, vertical, overhead etc., as the actual work.

On the North Western Railway in addition to the tests carried ou to determine the quality of the electrodes, monthly routine tests ar demanded from every operator ; and men, whose work falls below th standard called for, are taken off important welds. From these test it is found that reliance can be placed on the work of experienced opera tors.

Distortion and its correction.

Distortion of the structure during welding must be allowed for otherwise severe internal stresses will be set up.

With the extremely localised heat of the arc and the natural expansion of the heated weld, restrained as it is by the surrounding mass of cold metal, contraction on cooling takes place in the weld, resulting in distor tion. At first, seemingly unsurmountable difficulties are met with but, with experience gained, the erector soon learns how much to allow for distortion and expensive straightening after welding is avoided. A useful precaution is to see that wherever possible, the amount of deposit on either side of the neutral axis is kept as nearly equal as possible. In this way the distortion on one side will balance that on the other. Where this is impracticable, preliminary distortion of the work in a direction opposite to the welding effect is applied, and the contraction strains pulling the work straight relieve the internal stresses in the weld metal.

One method of eliminating distortion (not advocated by the writers) is that of " peening " the joint.

After each run of weld has been allowed to cool, it is lightly hammered with a round faced hammer. This operation is said to correct the distortion produced at each run of metal. It is claimed for "peening " that the run so hammered does not ultimately suffer in strength as any hardening that may have been induced by the hammering is corrected by the heat treatment and fusion of the subsequent run. The final run of metal is of course not " peened." Welding Engineers are at opposites over the advisability of " peening " and with the obvious danger of brittleness following the hammering and the doubt as to the correction by the following run, it is good practice to forbid the use of any but the light slag-removing chipping hammer.

ypes of welded joints.

In a girder structure, the types of joints generally used are as llows :—

1. the butt joint ;
2. the longitudinal fillet joint ;
3. the cross fillet joint ;
4. the angle fillet joint ;
5. The angle weld joint.

ıd

The several types of welds are often used in combination each with ıe other in fabricating welded joints. The design of these joints is ıown in detail in Figures 3, 4, 5, 6 and 7 (Plate I), the dotted sections ıowing the various runs of weld metal required to make up the joint.

Butt welds are used to transmit direct stress or longitudinal shearing tress or a combination of the two with or without bending or torsional ıoment having no component about the longitudinal axis of the weld. ;ee Fig. 10 (Plate II).

The included angle between the bevels of two plates being joined ɔgether by a butt weld is generally about 70° for plates up to $\frac{3}{8}$" thick nd 60° for larger thicknesses as the use of a more acute angle ıakes it difficult to ensure good welding at the apex of the Vee. For ılates up to $\frac{1}{2}$" in thickness, the bevel is cut in a single Vee from one ide ; but with thicker plates the joint is generally bevelled from both ides to avoid buckling of the plates due to contraction after welding. ;ee Fig. 3 (Plate I). Where possible, single Vee butt welds are finished vith a run of welding along the opposite surface. Sometimes butt joints ıre ground flush and cover plates with fillet welds are provided to develop ;0 per cent. of the strength of the plates being joined. Where cover ılates cannot be fitted, single " Vee " butt welds are reinforced with a leposition of 25 per cent. extra thickness of weld metal, and double " Vees " with 15 per cent. extra. The strength of a butt joint so reinforced is calculated on the thickness of the adjacent parent plate.

Fillet Welds are used to trasmit longitudinal shear or transverse shear or a combination of both with or without bending or torsional moment having no component about the longitudinal axis. See Fig. 11 (Plate II).

The cross fillet of a lap welded joint has a convex curved surface and approximates in shape to a quadrant or preferably to a quarter of an ellipse. The throat thickness should never be less than the width of the surface of contact in tension. See Fig. 13, 14 and 15 (Plate II). For a full fillet, the surface of contact in tension extends over the full thickness of the plate. The surface of contact in shear should have a width not less than 25 per cent. in excess of the throat thickness. See Fig. 9 (Plate II).

Side or longitudinal fillets are made with the width of both surfa in contact in shear on each plate equal. See Figs. 15 and 16 (Plate I

The size of a fillet weld is specified as follows :—

(1) In the case of a fillet composed of a single run of weld met by the gauge number of the electrode followed by the length of fillet la down by $16\frac{3}{4}$" of the electrode, the total length of the electrode befc use being 18". (This refers to No. 12 S.W.G. electrodes and larg smaller ones are made in 12 inch lengths).

(2) In the case of a fillet composed of more than one run of we metal, by the number of separate runs of weld metal laid down, ea separate run being specified in accordance with (1) above, or by the line dimensions of the surfaces of the fillet in contact with the parent metal.

Factor of Safety.

As the yield point of deposited weld metal more nearly approach the ultimate strength than is the case with structual steel, the safety fact (about 4) can be applied with equal confidence in places where the join are subjected to direct stresses.

Working stresses in Fillet Joints.

The sizes of fillet welds and the maximum allowable working stress laid down under the 1933 Rules of the Bridge Committee of the Railwa Board in India are as follows :—

Width of fillet weld in contact with parent metal.	Permissible working stresses in tons per lin. inch of fillet.	
	Shear.	Tension.
	Tons.	Tons.
1/4 inch	0·87	1/42
5/16 ,, ...	1·10	1/78
3/8 ,, ...	1·30	2/13
1/2 ,, ...	1·69	2/85
5/8 ,, ...	2·15	3/56
3/4 ,, ...	2·58	4/27

Working stresses in Butt Joints.

In welded Butt joints the maximum allowable working stress laic down in the same Rules as above is 5·70 tons in either tension or com pression.

In cases of field work under exceptionally difficult conditions, the factor of safety is increased.

Where these figures are not directly applicable, the stress on the lds is calculated on the following assumptions :—

The critical section of any weld is the longitudinal section containing e throat. Its area is throat × length and its section modulus is throat square of length divided by 6.

In case the resultant applied force is a central force without bend- g or torsion, the critical unit stress is either direct stress or shear, termined as follows :—

(*a*) Butt weld under direct force :—critical stress is direct (tension or compression) and is equal to total force divided by critical area.

(*b*) Butt or Fillet welds transmitting longitudinal shear :—critical stress is shear and is equal to total shear divided by critical area.

(*c*) Butt weld under direct force and longitudinal shear :—critical stress is either unit shear equal to total shear divided by critical area or unit resultant stress, equal to total oblique force divided by critical area and arbitrarily considered as direct stress. The critical stress is whichever one of these two unit stresses requires the larger section.

In case the applied force resolves into a central force plus bending or rsion, the extreme fibre stress due to bending and the extreme shear ue to torsion at each end of the critical section is determined by dividing ch moment by the section modulus of the critical section ; the maximum rect stress or shear thus determined is combined with the unit stresses ue to the central force per para. (*c*) above, the latter being considered niformly distributed. The critical stress is then either the maximum mbined unit shear or the maximum combined unit resultant (arbitrarily nsidered direct stress).

esign of welded joints.

The use of welded junctions enables the designer to place his joints xially to the members of the structure in a way which is at once much mpler than when designing with rivetted joints ; but, every welded int requires careful consideration in design to avoid serious concentra- ion of stress. Figs. 17, 18, 19 and 20 (Plate III) show typical junctions etween a diagonal fla subject to tension and a gusset. It will be seen hat the cutting of the corners of the flat has avoided all concentrations f stress at the edges of the flat. Figs. 21 and 22 (Plate III) show similar esigns for connecting an angle and a gusset. From these simple exam- les it will be understood that a welded joint requires just as careful lesign as a rivetted connection. Fig. 23 (Plate III) shows how the length f weld may be increased under conditions of severe loading.

Where possible, joints are so designed that the stresses on the welds nay be resolved into forms on which information on the strength of

welds is obtainable, *i.e.*, butt welds in direct compression or tension, fillet welds in end shear or hide shear. Fillet welds at joints are arran ed in such a manner as to prevent occurrence of bending moments twisting moments in individual welds about their longitudinal axes.

Where a member carrying direct stress is joined to another membe the centres of gravity of the welded seams should lie on the centre gravity of the members. See Fig. 12 (Plate II).

(*a*) For fillets of equal deposit the rule is :—

$$L_1 : L_2 = g_2 : g_1$$

(*b*) for fillets of unequal deposit the rule is :—

$$(a_1 \times L_2) : (a_2 \times L_2) = g_1 : g_1$$

These rules apply equally to members, the ends of which are c as shown in Fig. 21 (Plate III).

In compression joints and column splices, the welding is ma sufficient to transmit the full forces at the joint, except where the ends sections joined, are milled to fit, in which case, the welding is designed transmit not less than 50 per cent. of the longitudinal forces and t whole of any lateral forces.

But the most complete design giving perfeet axial conditions ceas to be exact when the loads and consequent strains take place in a struc ture. Thus moments are produced within welded joints and of thes very little is known. These bending or prying stresses within the wel cannot be neglected. A report of the Structural Steel Welding Com mittee of the American Welding Society states that this prying actio of welds is the more noticeable in fillets from $\frac{1}{4}''$ to $\frac{3}{4}''$. The calcula tion of these internal stresses is beyond the scope of the Structural En gineer, so in designing it is usual to provide joints that are capable o carrying the variety of stresses induced within them by taking value proved to be reasonably conservative by experiments. In some cases it is possible to provide weld metal in addition to that indicated by theory For example in designing cross fillet welds, it is usual to turn up th ends where possible. But where this is impossible the designer take into account 80 per cent. only of the length of the fillet when calculatin the strength of the joint. Single fillets are avoided because it is impossi ble to eliminate the prying stresses from such a joint. For this reaso fillet welds are used in balanced pairs. See Figs. 15 and 16 (Plate II).

Economical comparison with Rivetting.

In point of economy certain steel structures and bridges, which have been designed at the outset for arc welding, compare very favourably with those designed for rivetted joints. This point alone makes the future of the all welded structure an assured fact. On the other hand it is generally difficult to show an economy in using welding on a structure, which has been designed for rivetted connections. Nevertheless, as will be shown later, it frequently happens that a rivetted bridge may be strengthened or repaired more economically by welding than by rivetting.

rogress in Electric Arc Welding.

When in Europe in 1930, one of the authors (Mr. Everall) inspected any welded structures.

Two examples from Switzerland are of interest. One, an experiental 100 feet all-welded pony truss bridge carrying the main line railay traffic near Basle. In this bridge, no rolled steel sections have been sed and all parts have been made up of flat plates welded together, but ints having been introduced so as to make the test severe. After a eriod of some years this span is being removed and the welds examined id tested. As a safeguard, this bridge was built with a very small earance from the ground and during service it has been very closely atched and tested.

The other interesting example was a 130 foot deck type single ntersection Warren girder highway bridge over the Rhone. This ridge had been specially designed for welding ; and, even in 1930, such as the progress of electric arc welding as applied to bridges, that an conomy of 25 per cent. in weight had definitely been shown in comparion with a rivetted structure.

It was subsequent to Mr. Everall's visit to Europe that electric arc elding was first introduced into the bridge department of the North Vestern Railway and the following are examples of what has been done n this Railway in this new field of structural engineering.

ndus Bridge at Kotri.

(*a*). Roadway Brackets for the Reconstructed Indus Bridge at Kotri.

These were largely made up from second-hand steel plates and ngles recovered from other works on the line. The elevation of one of hese cantilever brackets (*vide* Plate IV) shows the fillet welds which ave been used instead of rivetting. The reason for this departure was hat the second-hand materials contained holes which made re-rivetting ut of the question.

These brackets constituted the first bridge work on this Railway where important stresses were carried solely on welds ; the former work aving been restricted to unimportant flooring plates on deck type spans, on which the staff gained valuable experience.

Strengthening 30 foot plate girders.

(*b*). The strengthening of Plate Girder spans by the relief of flange web rivet stresses on the Quetta Division.

These are 30 foot steel girders, which, put into the line in 1897, are being strengthened to carry M. L. Loads. Plates V and VI show the method of relief by welding. In this case, flats were rivetted to the web

and welded to the toes of the flange angles thus virtually transformin the angles from 4″ × 4″ × 5/8″ to 7″ × 4″ × 5/8″. It might be argued tha relief would have been achieved by welding the toes of the angles directl to the web but this might have endangered the web, which would hav been subjected to severe temperature stresses along a line where the stres would be high.

The method adopted follows the modern Australian and Britis practice. The work has been carried out under traffic without removin the spans from the line, the operator ceasing work a few minutes befor the passage of a train, so that the weld was allowed to cool before a loa came on to the bridge.

The comparison in costs of this method of relieving these rive stresses, and the alternative method of providing packings and cove plates rivetted to the angles and web showed an economy of 10 per cent in favour of welding. The idea of increasing the diameter of the rivet connecting the angles to the web was not practicable as stresses were to high.

A number of tests were carried out to ascertain the effectiveness o this method of strengthening. Specimen pieces of old girders wer welded using one, three and six runs of weld metal in the butt joint These were then cut up into lengths of 4″ (equal to the rivet pitch and tested in a tensile testing machine.

In the first series of tests the specimens (Test specimens No. 1) wer pulled with the butt welds on both sides in tension ; but in every case the specimens failed by the $\frac{7}{16}$″ plates tearing away at the rivets. Th results are shown in Table No. 1, Plate VI.

A further series of specimens were then prepared and the rivets cu out prior to subjecting the single butt welds to tensile tests. The result of these tests are shown in Table No. 2, Plate VI.

The figures given for the Breaking stress in shear have been inter-polated from the comparative strengths of fillet welds in tension and in shear.

As a result of these tests, 3-run welds were adopted giving a shear strength of 27 tons per 4 inch length of flange against an ultimate strength of only 21 tons for the rivet tearing away in the web plates.

Experimental All Welded Truss Purlins.

At the request of the Standards Office of the Railway Board, two all-welded truss purlins of 50 feet span such as might be used in a large station roof, where the number of columns interfering with valuable floor space is cut down to a minimum, have been made. These trusses are built up of light section mild steel angles and tees and are illustrated on Plates VII and VIII.

The design, prepared by Mr. Swain, Assistant Controller of Stand-ardization, embodies extreme lightness with strength and is an example of a structure, which could not have been made so lightly without th

ntroduction of welding. Details together with the stresses induced in he weld connections under a uniformly distributed load of 3 tons are hown in Plate VIII.

These 50 feet purlins weighed 8·40 cwt. each or less than 19 lbs. per oot run and the cost complete was about Rs. 12/- per cwt., a figure which vould be reduced to about Rs. 10/- per cwt. in the event of a number of russes being built in mass production, such as would be required for a arge roof.

The welding of one truss was accomplished in ten hours by one »perator using 150 No. 10 gauge Mild Steel Electrodes.

These trusses, designed to carry a uniformly distributed load of 3 ons, were tested to destruction ; the yield point being reached when a ıniformly distributed load of 110 cwts. had been applied and complete 'ailure occurring under a load of 150 cwts.

Failure was entirely due to the buckling of the angles under com- »ression and not one single weld showed any sign of fracture. Plate VII ;hows one of these trusses before and after failure.

Replacing the Bottom of a large High Service Water Tank at Lalamusa.

When the bottom of one of the rivetted steel plate water tanks at Lalamusa was found to be very badly corroded and replacement became necessary, several schemes were considered as to the most efficient and economical method of carrying out the repairs.

The substitution of new pressed steel buckle plates was out of the question as none of the standard sizes would fit and the cost of pressing special plates would have been prohibitive.

Eventually the schemes were reduced to two, namely the provision of a new bottom consisting of steel plates with rivetted lap or butt joints or a similar plate bottom resting on an angle rivetted to the sizes of the tank and having all joints welded. The latter or welded bottom obviated the caulking and lapping of rivetted joints and for these reasons was chosen on point of economy.

The work of replacing the bottom of this water tank is now in hand and is expected to be completed for the very low cost of Rs. 2,100/-.

The design is shown in Plate IX.

The Authors are indebted to Mr. N. Hackney, B. Sc., Metallurgist to the North Western Railway, who so kindly made the photomicrographs and carried out many of the tests in his laboratory. And in conclusion they wish to thank the Administration of the North Western Railway for allowing them to present this paper.

PLATE I

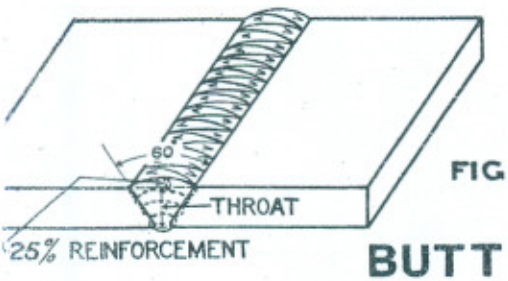
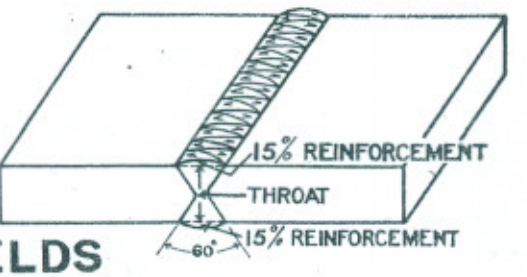
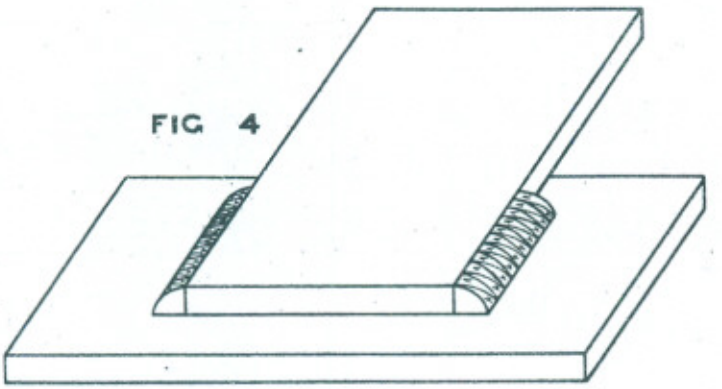


FIG 3



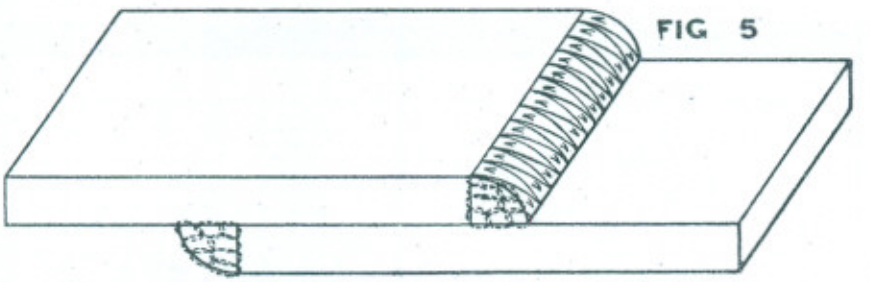
BUTT WELDS

FIG 4



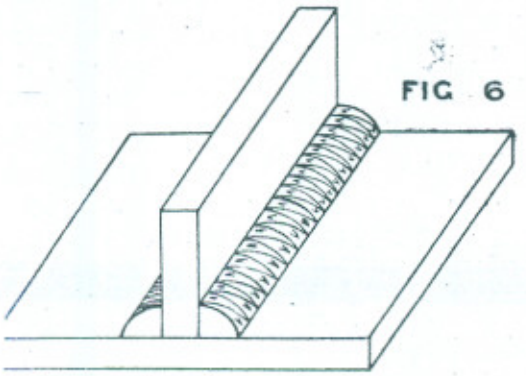
LONGITUDINAL FILLET WELDS

FIG 5



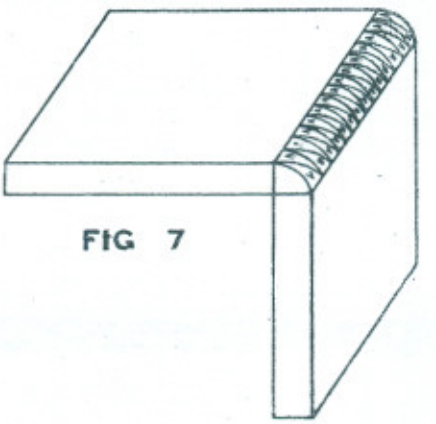
TRANSVERSE OR CROSS FILLET WELDS

FIG 6



ANGLE FILLET WELDS

FIG 7



ANGLE WELD

PLATE II

FILLET WELDS

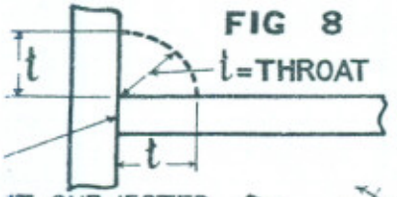


FIG 8

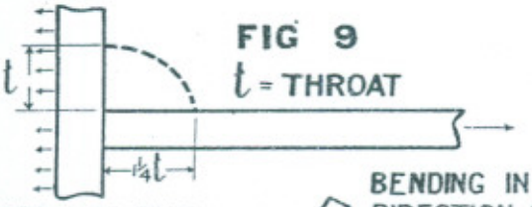


FIG 9

T SUBJECTED
LONGITUDINAL
AR

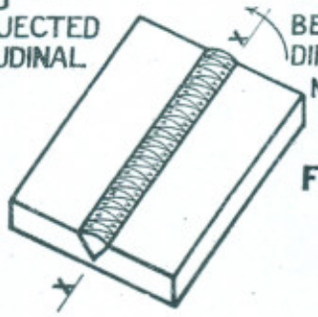


FIG 10

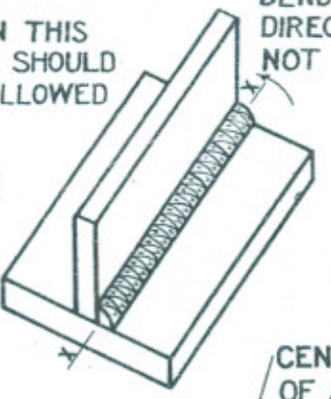


FIG 11

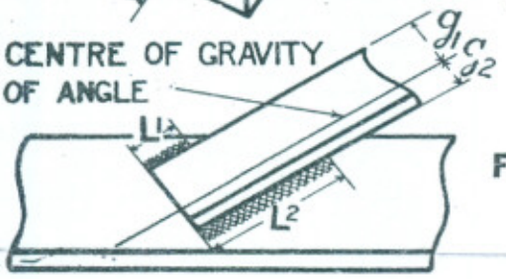


FIG 12

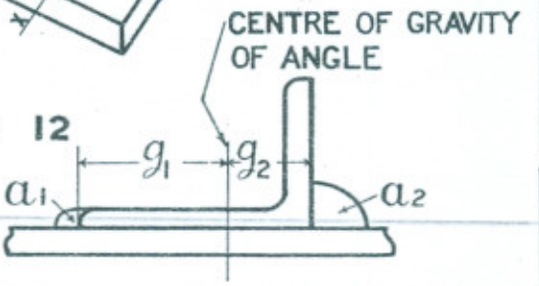


FIG 13

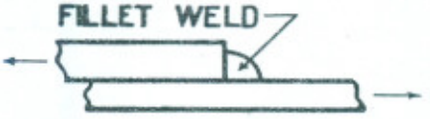
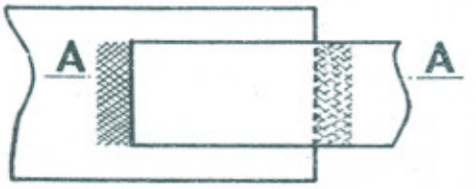
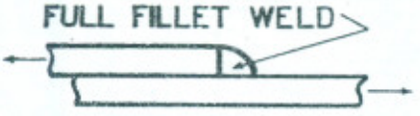


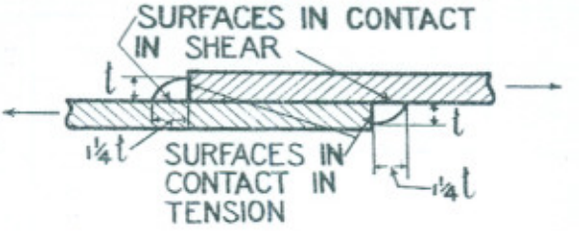
FIG 15



SECTION ON AA

FIG 14





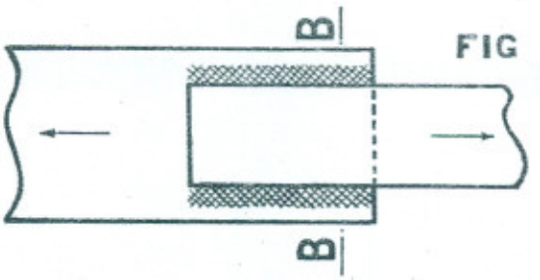
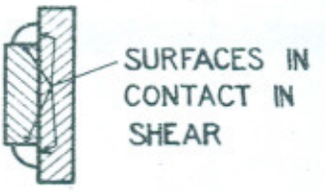


FIG 16



SECTION ON BB

PLATE III

FIG 17

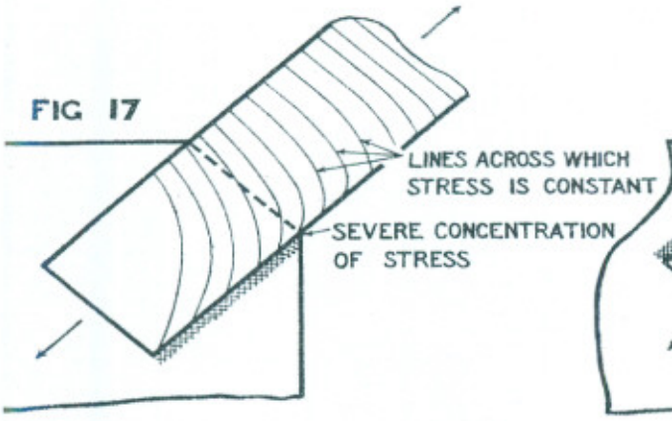


FIG 18

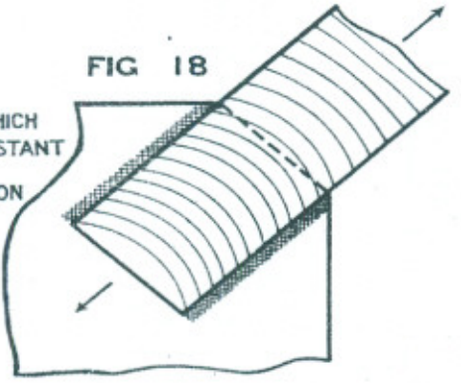


FIG 19

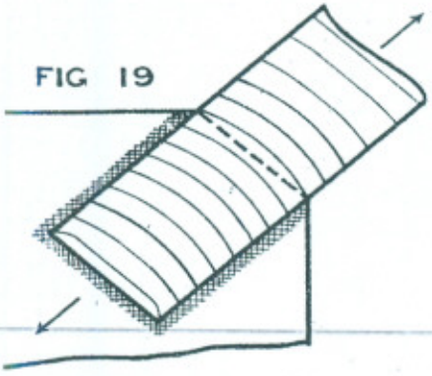


FIG 20

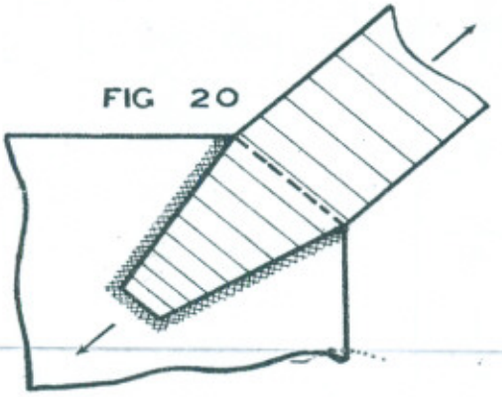


FIG 21

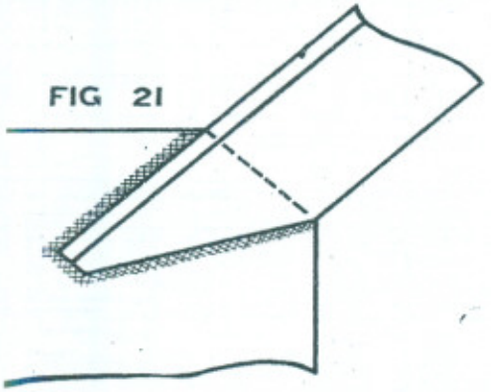


FIG 22

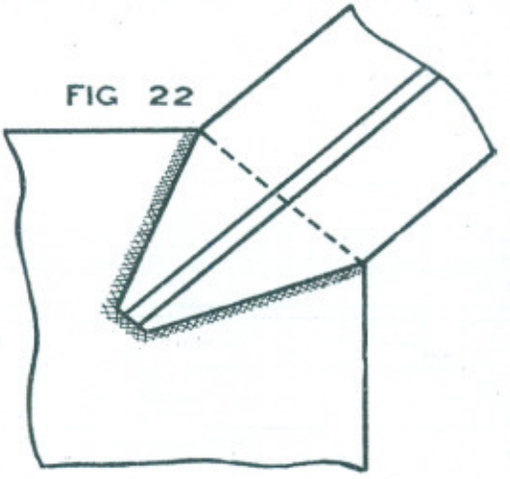


FIG 23

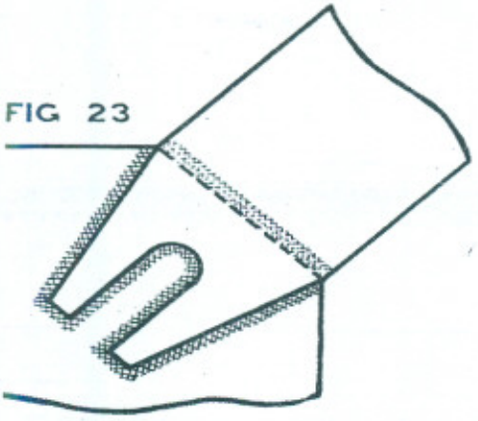


PLATE IV

SECTION ON D.D.

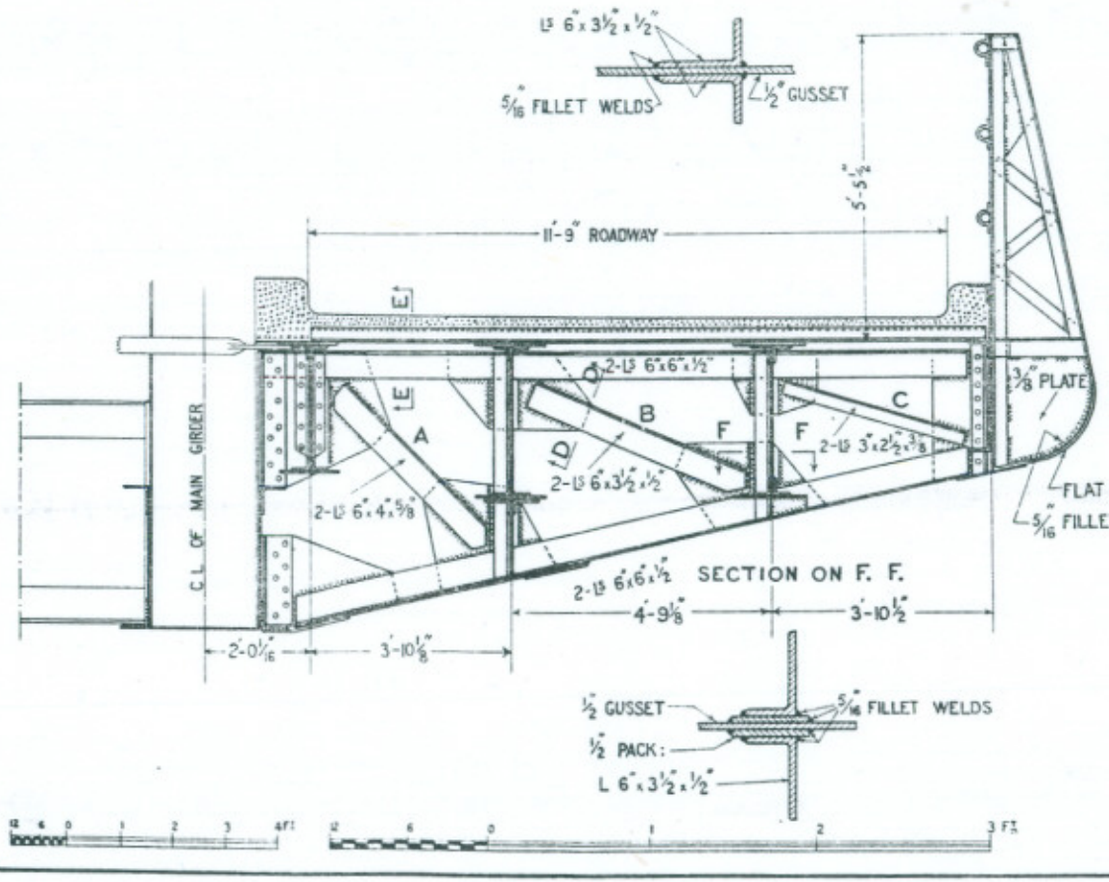
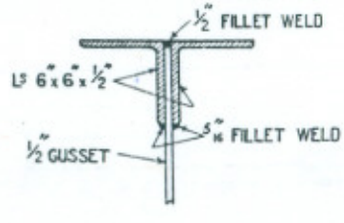


TABLE OF STRESSES IN WELDED CONNECTIONS

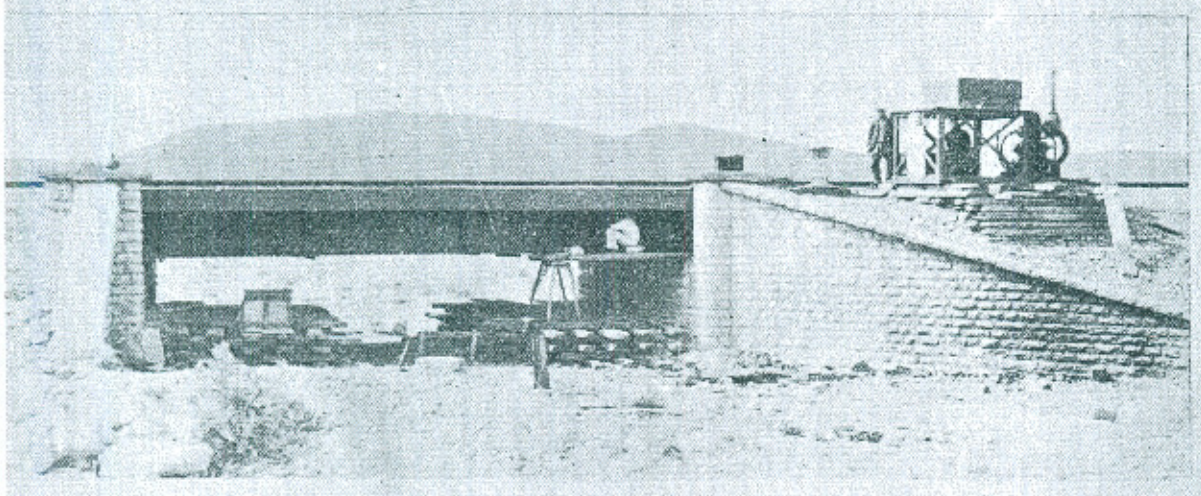
DIAGONALS	MAX: STRESS	TOTAL LENGTH OF FILLET IN INCHES	SIZE OF FILLET	MAX: STRESS PER LINEAL INCH OF FILLET	PERMISSIBLE STRESS PER LINEAL INCH OF FILLET
A	45·5 TONS	51 INCH	5/16"	0·89 TON	1·10 TON
B	45·5 "	54 "	"	0·84 "	"
C	21·0 "	30 "	"	0·70 "	"

ENTIRE STRESSES CARRIED BY LONGITUDINAL SIDE FILLETS ONLY.

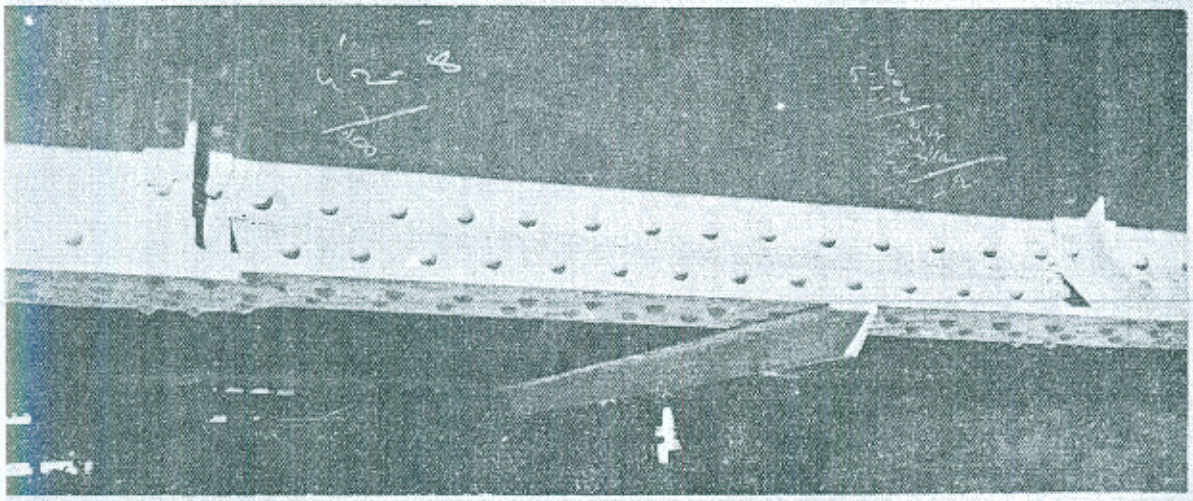
SECTION ON E. E.



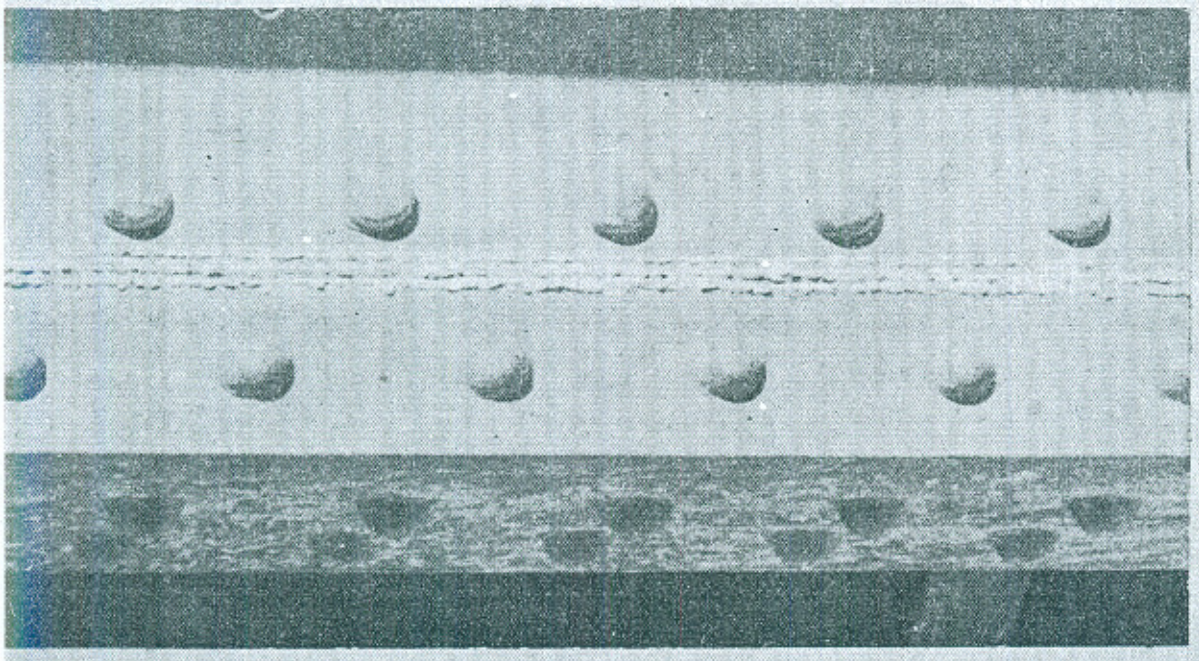
)O F$^{T}_{H}$ SPAN ON SIBI KOLPUR SECTION PLATE V



GENERAL VIEW SHOWING WELDING PLANT



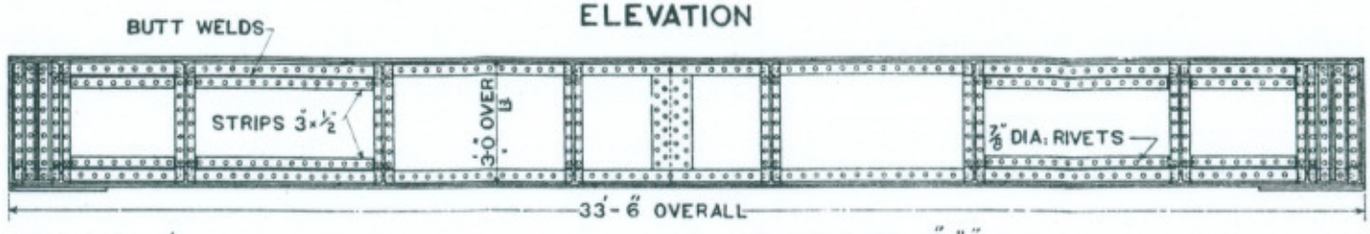
LOWER FLANGE AFTER STRENGTHENING



_DING BETWEEN NEW FLAT AND FLANGE ANGLE

PLATE VI

ELEVATION



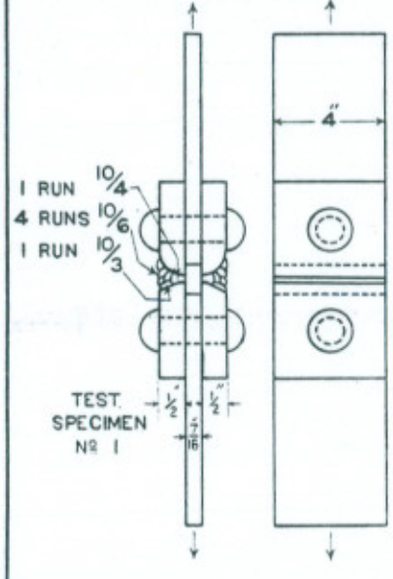
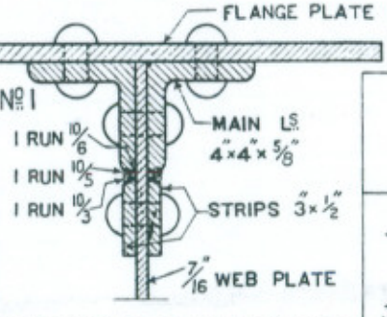


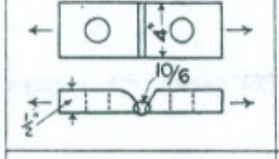
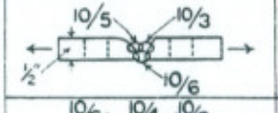
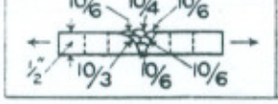
TABLE OF TEST RESULTS Nº 1

NUMBER OF RUNS OF WELD METAL	BREAKING LOAD IN TONS
1	21·0
3	,,
6	,,
ALL SPECIMENS FAILED BY TEARING OF 7/16 WEB PLATE.	



SECTION THROUGH TOP FLANGE

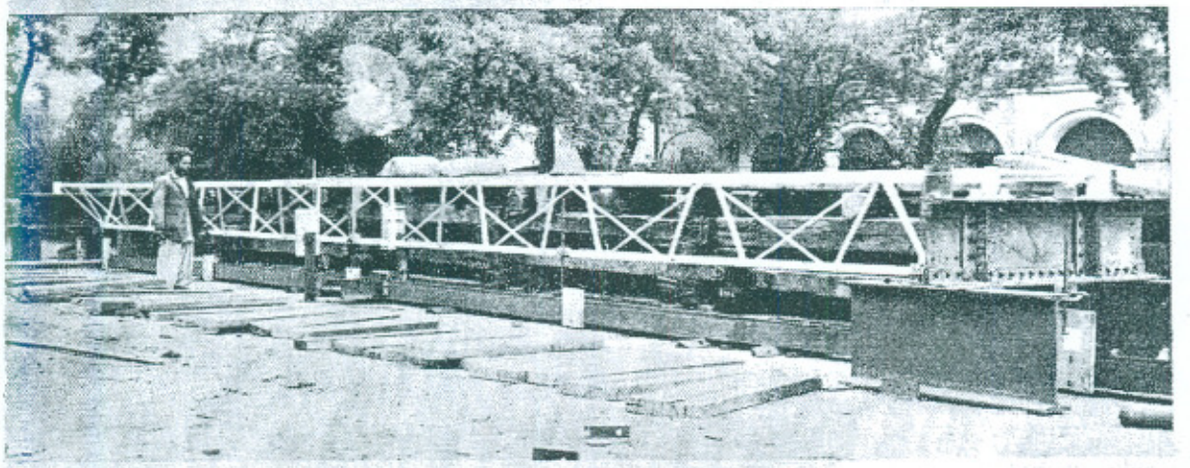
TABLE OF TEST RESULTS Nº 2

SPECIMEN	NUMBER OF RUNS OF WELD METAL	BREAKING STRESS IN DIRECT TENSION	BREAKING STRESS IN SHEAR
	1	16·7 TONS	12·5 TONS
	3	22·0 ,,	16·5 ,,
	6	34·2 ,,	25·6 ,,

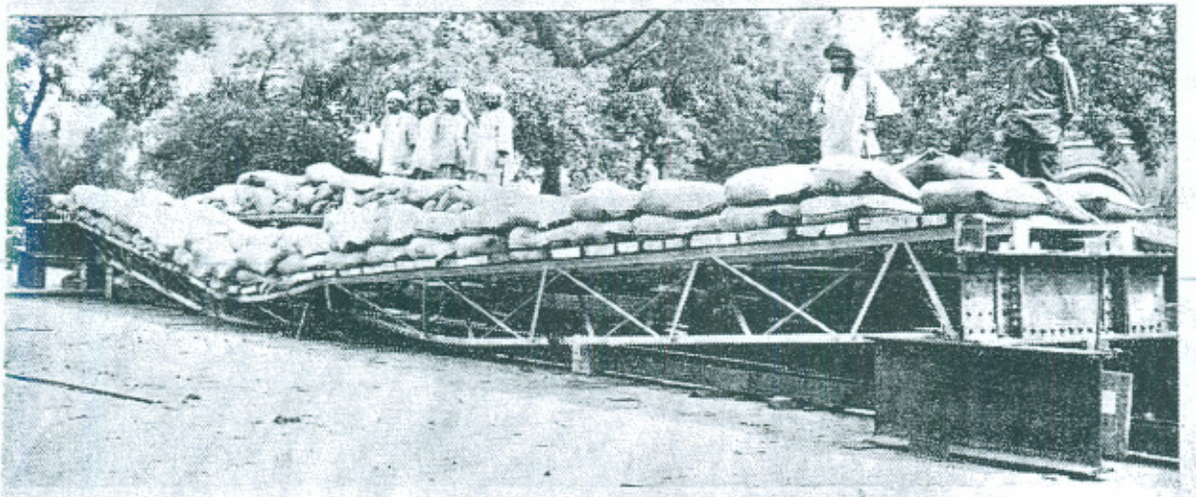




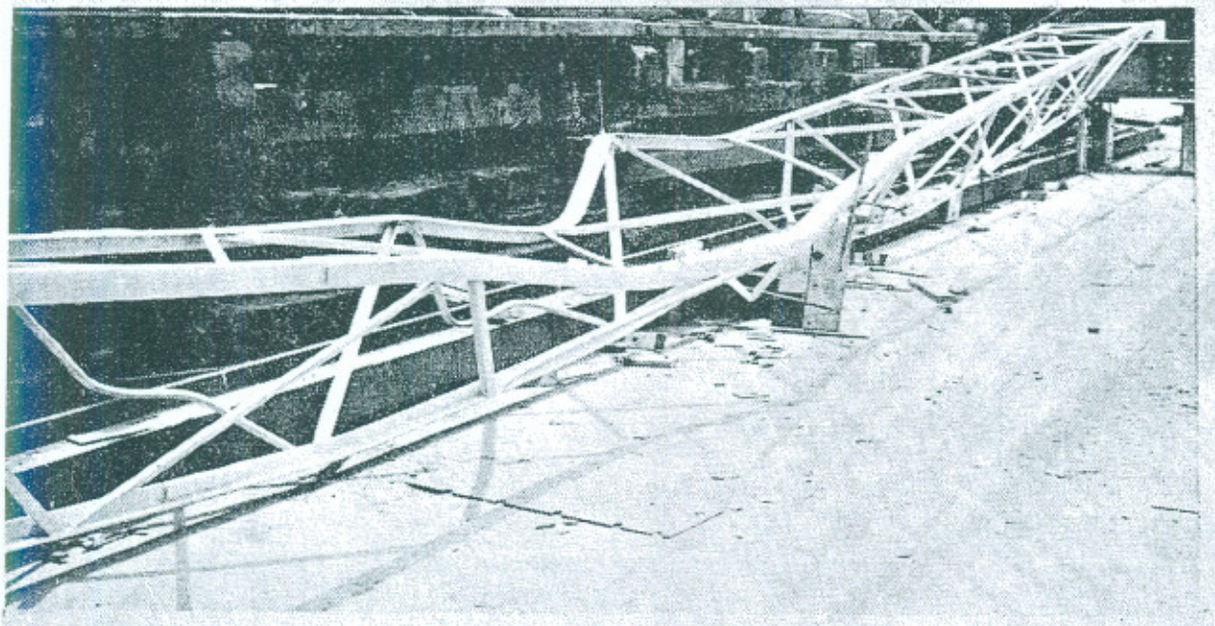
ALL WELDED TRUSS PURLIN PLATE VII



BEFORE TEST



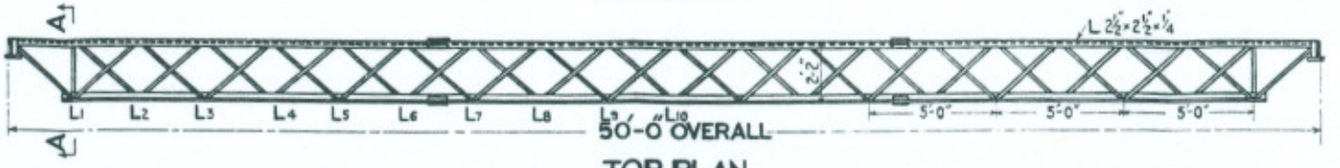
AFTER FAILURE UNDER TEST LOAD



DETAIL AFTER FAILURE

PLATE VIII

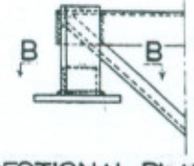
ELEVATION



TOP PLAN



ELEVATION



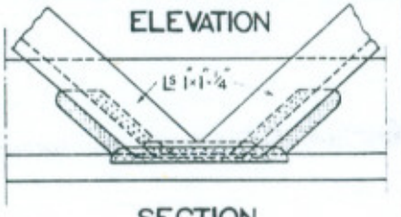
SECTIONAL PLAN ON B.B.



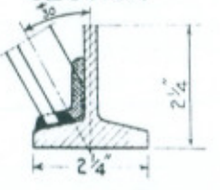
SECTION ON A.A.

DETAIL SHOWING DIAGONALS WELDED TO BOTTOM BOOM

ELEVATION



SECTION



ALL CONNECTIONS MADE WITH ¼ IN FILLET WELDS – 1 RUN 10/7

TABLE OF STRESSES IN WELDED CONNECTIONS

MEMBERS	MAX. STRESS	TOTAL LENGTH OF ¼" FILLET IN INCHES	STRESS PER LINEAL INCH OF ¼ WELD FILLET IN SHEAR	PERMISSIBLE MAX. STRESS PER LIN INCH OF FILLET IN SHEAR
U0 L1	1·42 TON	5·6	0·25 TON	0·87 TON
U0' L1	1·28 "	"	0·24 "	"
U1 L2	1·14 "	"	0·20 "	"
U2 L3	1·00 "	"	0·18 "	"
U3 L4	0·86 "	"	0·15 "	"
U1 L1	1·00 TON	3·5	0·29 TON	0·87 TON
L1 U2	1·28 "	5·6	0·24 "	"
L2 U3	1·14 "	"	0·20 "	"
L3 U4	1·00 "	"	0·18 "	"
L4 U5	0·86 "	"	0·15 "	"

PLATE IX

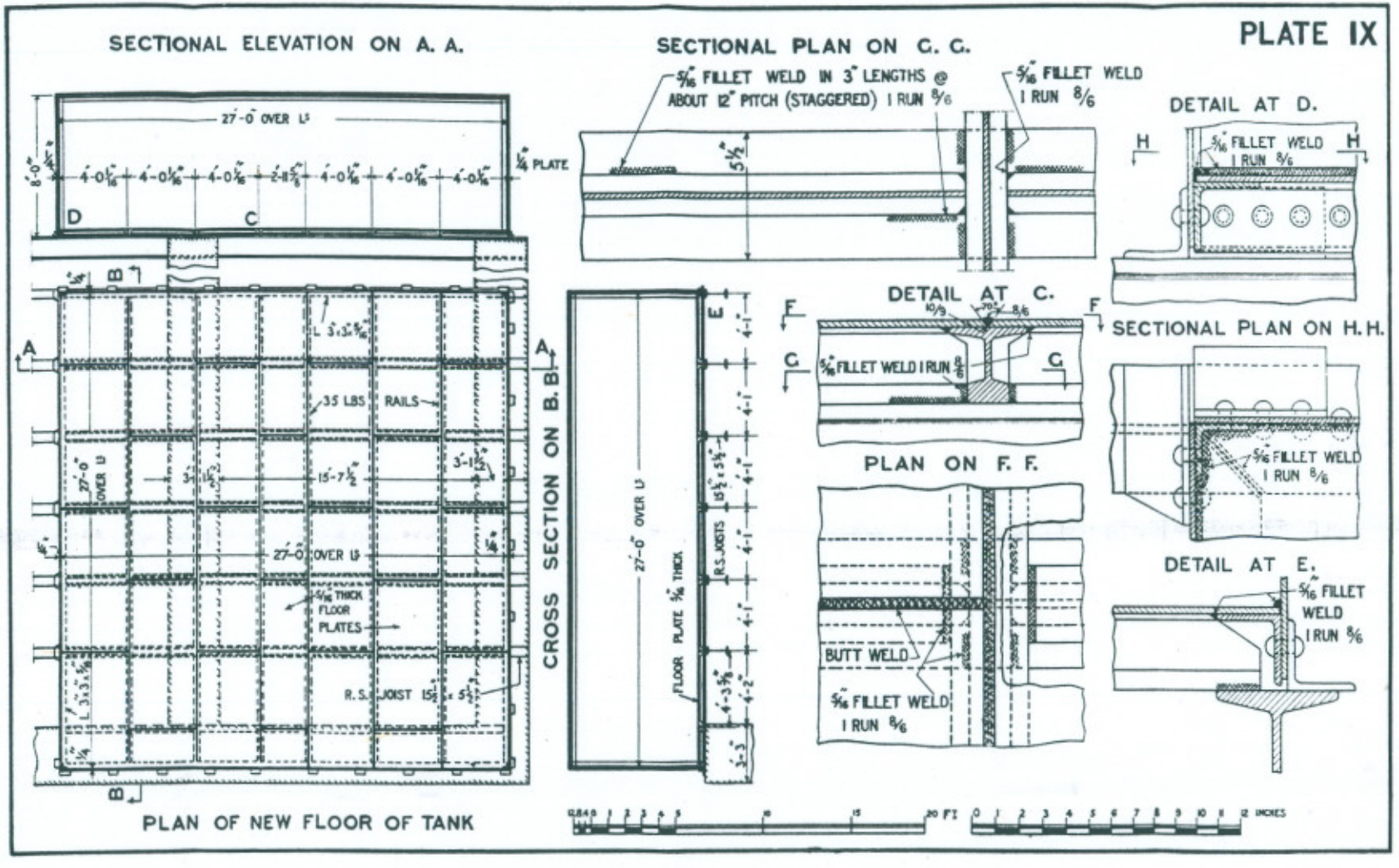
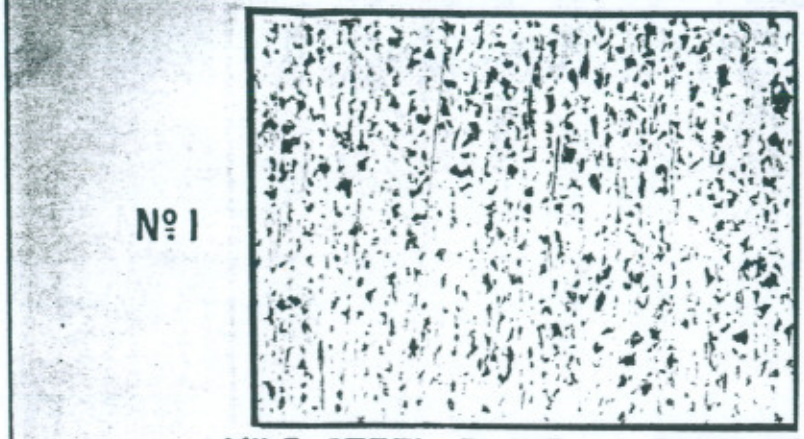

SECTIONAL ELEVATION ON A. A.
SECTIONAL PLAN ON C. C.
5/16 FILLET WELD IN 3" LENGTHS @ ABOUT 12" PITCH (STAGGERED) I RUN 8/6
5/16 FILLET WELD I RUN 8/6
DETAIL AT D.
H
5/16 FILLET WELD I RUN 8/6
27'-0" OVER L's
8'-0"
4'-0 1/16" 4'-0 1/16" 4'-0 1/16" 2'-8 5/8" 4'-0 1/16" 4'-0 1/16" 4'-0 1/16"
1/4" PLATE
D
C
B
A
L 3"x3"x5/16"
35 LBS RAILS
27'-0" OVER L's
15'-7 1/2"
3'-1 1/2"
5/16" THICK FLOOR PLATES
R.S. JOIST 15"
5 1/2"
CROSS SECTION ON B. B.
E
4'-1"
15 1/2"
5 1/2"
R.S. JOISTS
4'-3 5/8"
4'-2"
3'-3"
FLOOR PLATE 5/16" THICK
DETAIL AT C.
F
C
5/16" FILLET WELD I RUN 8/6
SECTIONAL PLAN ON H.H.
5/16" FILLET WELD I RUN 8/6
PLAN ON F. F.
BUTT WELD
5/16" FILLET WELD I RUN 8/6
DETAIL AT E.
5/16" FILLET WELD I RUN 8/6
PLAN OF NEW FLOOR OF TANK
0 1 2 3 4 5 10 15 20 FT
0 1 2 3 4 5 6 7 8 9 10 11 12 INCHES

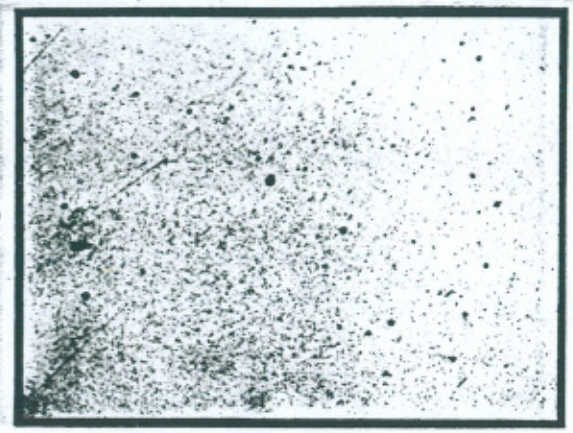
PHOTO - MICROGRAPHS OF A TYPICAL WELD x 100

PLATE X



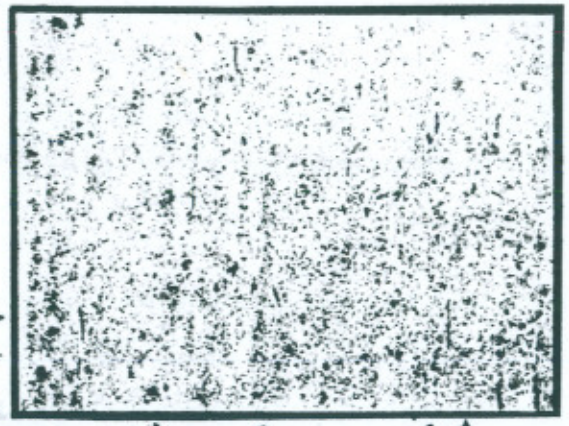
Nº 1

MILD STEEL PLATE AS RECEIVED



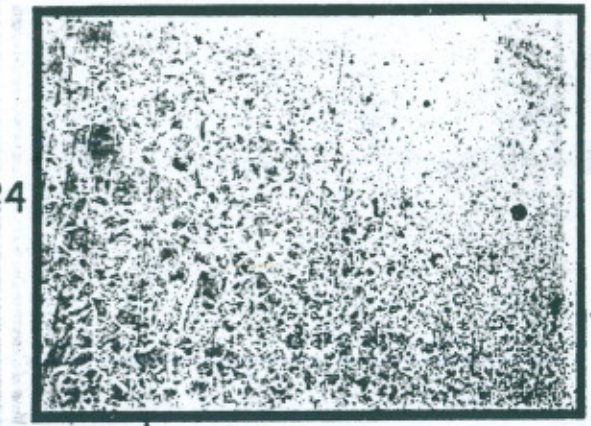
Nº 2

DEPOSITED WELD METAL



Nº 3

PARENT METAL



Nº 4

WELD METAL

1500°C

TEMPERATURE RANGE — 720°C — 900°C — 900 TO 1500°C

ENLARGED CRYSTAL STRUCTURE

OVER-HEATED STRUCTURE

DISCUSSION.

Mr. W. T. Everall in introducing his paper said that he wished to mention that the object in writing it had been to place before the Members some of the factors which Engineers have to consider when dealing with this branch of engineering, and particularly for Engineers who, like themselves, have had to gain their knowledge and experience from a study of some of the more elementary principles employed in the practice of welding.

On the North-Western Railway, Electric Arc Welding of certain parts of bridges had only been made use of for the past three years, he felt confident however that as their knowledge and experience of the subject increased, much wider fields for its use would be developed in the future.

In these days Electric Welding was employed in every industry that used steel sections. It was extensively employed on the construction of steel framed buildings, railway and road bridges, points and crossings for railways and tramways, and it had also entered largely into the construction of modern ships.

It was replacing the use of rivetted work for many different types of structures and effecting considerable economies in cost and in the weight of metal used.

Arc Welding to be successful, required considerable technical knowledge and skill both in the preparation of designs and in the application of the welds and in maintaining full control by metallurgical tests and skilled supervision.

The Author mentioned that Mr. Berridge who was jointly responsible for that paper had done a great deal of the work for the paper in describing the detailed features of welding and in designing many of the welded parts now being introduced.

Mr. Berridge when on leave in 1932, took a special course of instruction on Welding in England and had brought himself up-to-date with some of its latest developments and practice. Mr. Berridge would have something to say to the Members on several matters connected with the technical features of welding which had not been dealt with fully in the paper.

Mr. Everall further remarked that he was very fortunate in having the valuable assistance of Mr. Hackney, the Chief Metallurgist and Analytical Chemist of the North-Western Railway. He felt that they could not have made any definite progress in welding or have been able to place much reliance on such work without his assistance. He had solved many difficult problems in connection with different conditions of welding and in determining the various chemical properties and mechanical strengths of weld metal.

Mr. Hackney's work had enabled them to fully appreciate the value of the welded joint, and what was more important, its limitations. He wished to express his great appreciation of what Mr. Hackney had done

in that connection and hoped that the Members would have the privilege of hearing some of his views on the subject in the discussion.

Another feature which was of primary importance in connection with welding was in finding and training the right type of man for the work. The Author further remarked that they have been successful in training some of the Punjabi workmen for that purpose and they had now attained a degree of skill and efficiency which enabled them to rely upon their work in producing an acceptable standard. This did not however obviate the necessity for constant live supervision. The work done by the welders was tested frequently by means of routine tests taken several times during each month. Sometimes test specimens were cut out of the work and at other times by making up special test specimens similar in character to the work being operated on. Examples of test specimens were referred to on Plate No. VI of the Paper.

The paper only described several typical welded works and did not by any means cover the full range of their uses in the Engineering Branch of the North-Western Railway. Their uses on several works had effected economy both in money and time and in avoiding costly alternatives. For example quite recently a scheme was put forward where an estimated saving of practically Rs. 30,000/- could be affected on a 1½ lakhs job. That was in connection with the insertion of stepped bearings under the main girders of about 200 bridge spans in the Main line. The use of welding had obviated the necessity of lifting the girders out of the line and putting them back again. The scheme enabled them to attach new stepped bearings and bracing members to the bridge *in situ* by means of welding. Another example was in the welded truss which was illustrated on Plates VII and VIII of the paper. That particular form of truss had been adopted and incorporated in a standard type of station platform roof for use on railways all over India. Their economy was considerable as such a structure could be built, for approximately 75% of the cost of a similar structure where rivets were used.

Mr. P. S. A. Berridge in opening his part of the discussion on the paper corrected the following errors which had been observed in the text.

On page 81, in the Table giving the Strength of Weld metal produced from various makes and types of electrodes, the Ultimate Strength of specimens made with M. W. P. Cresta Asbestos Covered Electrodes should be 29·5.

On page 84, the number of Portable Welding Sets in the Bridge Department had been given as five. This should have been four only. Also the speed of the Diesel power units was erroneously given as 1,450 R. P. M. It should have been 1,200 R. P. M., which was the speed of the Gardiner engines employed in the department.

On page 88, the permissible working stresses given in tons per lin-inch of fillet subject to Tension should be read as 1·42, 1·78, 2·13, etc. instead of 1/42, 1/78, 2/13; etc.

And on the same page in the paragraph on Working Stresses in Butt Joints, the maximum allowable working stress should read 5·70 tons per square inch and not just 5·70 tons.

Finally on page 90, the rule (*b*) for the design of joints should read as follows :—

$$(a_1 \times L_1) : (a_2 \times L_2) = g_2 : g_1$$

The speaker then proceeded with the discussion. He wished to mention some further test work which had been carried out to determine the strength of longitudinal and transverse fillet welds. After that he would say a word on the cost of welding as compared with rivetting and finally he would offer some explanation of the microscopic structure of the metal at a welded joint.

Dealing firstly with the tests on longitudinal and transverse welds, the speaker said that a very large number of tests had been carried out to determine the relative strengths of such joints made with various sizes of welds. The welding of these test specimens had been done by one of the Punjabi welders in the Bridge Workshop in Jhelum and the specimens, of which there were over 300, had been broken on the 50 ton Buckton Testing Machine in Mr. Hackney's laboratory in Moghalpura. To keep the breaking stress within the capacity of this machine and at the same time to avoid introducing non-axial forces on the welds, the length of the transverse fillets had had to be limited to four inches and the longitudinal fillets to only two inches. The accompanying sketches and graphs (Plate XI) showed the forms of these test specimens and the results, the ordinates being the breaking stress and the abscissae the sizes of the welds. The results of these tests were particularly interesting as they confirmed the figures given in the Railway Board 1933 Rules for the permissible working stress in welds made under shop conditions subject either to shear or to tension, a factor of safety of about four having been allowed.

The writers had dealt with the generating plants, which had been put into use in the Bridge Department of the North-Western Railway and the speaker wished to point out that the consumption of lubricating oil for the Petrol driven machine had been shown as 1 gallon per 9 hours running. That was a mistake, the correct consumption being about a quarter of that amount making the total running cost about Rs. 12/- for laying down 48 feet of single run of 10/5 weld—the work done in 9 hours.

Not having had the opportunity of making any all-welded structures, other than the special truss purlins illustrated in Plates VII and VIII, the speaker regretted being unable to give any comparative figures of the costs of rivetted and all-welded bridges. The cost of these all-welded truss purlins as made in Jhelum worked out to Rs. 200/- per ton as against Rs. 260/ per ton for a similar rivetted structure, the weight of which would necessarily be at least 20 per cent. more owing to the provision of gussets, cleats, etc.

The final cost of a completed umbrella roof of an island platform with the columns placed at intervals of 50 feet and two all-welded purlin trusses used to carry a roof of corrugated iron sheeting was found to be Rs. 1/3/- per square foot of area covered as compared with the cheapest rivetted roof structure costing about Rs. 1/10/- per square foot covered.

The speaker considered that some figures of the cost of welding used in strengthening existing rivetted girders might be of interest.

From actual observations taken on the work of strengthening the 30 feet spans on the Sibi-Kolpur Section shown on page VI, it was found that in the average day's work (9 hours) the operator laid down 130 No. 10 Gauge Electrodes or 15·6 feet of completed weld of 3 runs and the cost of this day's work came to—

Running Petter Oil driven plant, fuel, etc.	..	Rs. 4/4/-
Labour, *i.e.*, Driver, Welder and Cooly	..	Rs. 4/-/-
Electrodes at Rs. 72/- per 1,000 feet	..	Rs. 14/-/-
Total for 15·6 feet weld	..	Rs. 22/4/-

or just under Rs. 1/8/- a foot run.

Since the paper went to Press, tenders had been accepted for the supply of three new Single Operator twin cylinder Diesel sets with which they hoped to further reduce the cost of welding.

Another interesting work of which the cost of using welding showed up very favourably was the fabrication of all-welded mild steel rocker-cum-roller bearings for some 150 feet spans on the North-Western Railway. Ordinarily such bearings were made of cast steel, the cost being about Rs. 450/- per ton. The cost of the welded mild steel bearings was less than Rs. 350/- per ton. Those bearings were even now being made up in the Bridge Workshop at Jhelum.

Mr. Everall had referred to the training of Punjabi labour in the art of welding. The ordinary straightforward flat welding was relatively simple and with a little practice a reliable weld could easily be put down ; but, to be really proficient at overhead welding, it took a prospective welder years of practice before he could be relied upon. That fact naturally had a restricting influence on the use of welding and until the welders became more experienced, the use of overhead welds was being avoided altogether. Another factor against the use of overhead weld was that special Electrodes had to be used and they were about 28 per cent. more costly.

On page 86, paragraph on Distortion and its correction, the Authors had mentioned, but not advocated, a method of eliminating distortion by "Peening." That really was cold working a weld, a process which raised the tensile strength but lowered the ductility. It was interesting to note that a recent collapse of an all-welded oil tank in U. S. A. had been entirely due to the workmen peening or caulking a welded seam.

The Author mentioned that he had been anxious to consider the microscopic structure of the metal at a weld from the Metallurgist's point of view. A diagram (Plate XII) had been prepared to explain the changes which took place in the microscopic structure of ordinary structural mild steel (0·2 per cent. carbon content) when it was heated and subsequently cooled as happened in Electric Arc Welding.

At ordinary atmospheric temperature structural steel consisted of Ferrite and Pearlite as shown in Plate X, Photo-micrograph No. 1, where the microscopic structure of such steel was shown as it was when received from the mills.

Ferrite was what Metallurgists called α iron. It had certain properties which were totally different from other allotropic forms of iron. α iron existed in steel of 0·2 per cent. carbon content up to the lower critical temperature Ac 1. Above the upper critical temperature Ac 3, iron existed in the form known as γ iron.

The speaker then considered the changes as the steel was heated. Up to Ac 1, the iron existed as α iron in which carbon was insoluble. Between Ac 1 and Ac 3, that α iron changed to γ iron in which carbon was soluble up to 2 or 3 per cent. At Ac 3, a new crystalline system formed and carbides dissolved and diffused. The process of crystal growth and diffusion of carbides increased until the steel became liquid.

In cooling, crystals formed as the steel became solid and continued to grow until the temperature dropped to AR 3, *i.e.*, the upper critical cooling temperature. At AR 3, crystal growth ceased and between AR 3 and AR 1, γ iron changed to α iron, the enlarged crystals remaining but altering their properties and the carbon being thrown out of solution.

Thus, under the microscope, steel which was heated and cooled at a weld was transformed from a homogeneous microscopic structure to one of enlarged crystals, as shown in Photo-micrograph No. 4.

At a weld, the temperature gradient during fusion was very steep. From the comparatively carbonless electrode deposit at the surface down to the unaltered parent metal there was a succession of layers of metal of varying metallurgical structures resulting from the metal having been heated to various maximum temperatures. There was a line of demarcation between the comparatively carbonless deposit and the carbon-containing parent metal. This parent metal which had been heated to above 900°C produced a zone of crystals of progressively diminishing size, very large at the surface nearest to the fusion and very small at the zone at which crystal growth only just started. This zone of large crystals was of great importance as it was there that fatigue cracks developed very rapidly. Below the 900°C zone and above 720°C there was a zone of fuzzy outlines and indefiniteness. The original crystals had been heated above Ac 1 and had therefore begun to dissolve carbon ; but, as the temperature ceased to increase, this process had not been completed and there was a region of high carbon percentage at the surfaces of these crystals and no carbon at their centres with gradations in between.

That explained why it was that welded joints so often failed along the line where the deposit metal joined the parent metal. The speaker was referring of course to good welds where the added metal had been properly fused with the parent steel.

The reason why enlarged crystals were liable to comparative early failure by fatigue was that fracture, taking place along the octahedral slip planes of the crystals, created a greater stress on the adjacent crystals if those crystals were enlarged than it did with small crystals.

The depth of these various layers of metal at a welded joint was governed by the amount of heat used to fuse the weld, which incidentally was relative to the current used.

The usual commercial method by which the initial strength of the steel could be restored after welding was by normalising, *i.e.*, by heating the whole metal to 900°C very slowly and very evenly so as to avoid strains and to enable carbides to diffuse adequately. Then the whole metal would be cooled in air slowly and evenly. This process re-crystallised the whole metal uniformly and at about 900°C the metal became entirely γ iron with crystals only just beginning to grow from their molecular sized nuclei. If now the temperature did not exceed 900°C crystal growth was negligible and as the temperature started to fall down at once, growth ceased and as it fell below AR3, that homogeneous fine structure was preserved giving back the steel at nearly its initial strength and toughness. But as a bridge girder could not be normalised, until welding research carried them a stage further, engineers must bear in mind the " damage " done to steel by electric fusion welding.

The speaker said that he did not for one minute wish to create an impression that a good weld could not be relied upon, but he did wish to impress engineers that welds applied to bridge girders did not have the same ability to withstand fatigue stresses as did the ordinary parent metal.

The speaker mentioned that he had used the word " damage " in connection with the effect on steel by welding ; incidentally he considered it only fair to state that rivetting also " damaged " the steel in the same sense of the word, because it localised stresses.

In conclusion the speaker said that he would like to express his thanks to Mr. Hackney who had so kindly helped him to understand the changes in the microscopic structure of the metal at a weld. He said that he was not a Metallurgist and without Mr. Hackney's assistance he would never have been able to put up this interesting data before the Members.

Mr. Everall had credited him with the majority of the work in preparing the paper and he wished to express his thanks to Mr. Everall for allowing him to be a joint contributor. It had been a great pleasure to be associated with Mr. Everall in that work.

Lt.-Col. W. Macrae remarked that metallic arc welding is a modern development that has come in since most of us finished our technical training, and that it is just spreading into India.

This paper gives its present state of development in certain lines hat are applicable to Indian steel structural conditions.

On the railway we have also been trying this metallic arc welding on ·ails and on points and crossings, but are not yet in a position to report lefinitely on these trials.

Apart from bridge structures this process is being brought in for ›uildings, such as platform sheds, where a large saving can be obtained ›y welding in place of rivetting.

The important point is the need of skilled reliable workmen under ;ood supervision ; the latter should, and on the North-Western Railway loes, include help by a Metallurgist.

So far the North-Western Railway have worked on very conservativc ines as regards metallic arc welding but even so have been able to carry ›ut strengthening of structural steel work that would really have been mpracticable by rivetting and thus have saved replacement. This is a ield in which most branches of engineering are concerned.

MR. N. HACKNEY said that as a despised theoretical man, he felt ·ather out of place in that assembly of practical experts. The speaker said that Mr. Everall had spoken kind things about his share in working out the data embodied in the paper, but his part was relatively easy, consisting as it did of little more than making a number of physical tests of welds made by the department.

Normally, his dealings with mechanical brethren were closer than they were with Civil Engineers and he was afraid he had acquired with them the wholly undeserved reputation of being a confirmed anti-welder. The two main branches of the Engineering profession seemed to him (very broadly speaking, of course) to be concerned with two different types of stresses ; the mechanical engineers dealt with moving pieces of metal, whereas the structures such as bridges, dams, pipelines and so on, stand still, or should do. Axles, connecting-rods and springs were subjected to everlasting alterations of stress : bending, push-pull, or torsional, and the effects of welding on parts such as those were very definitely adverse, whereas it might increase the strength of a bridge, for example, where the chief stresses were static compression or tension. In the one case the essential property of the material, as far as its design was concerned, was its Fatigue Limit ; in the other it was its Yield Limit.

The speaker said that he had been surprised on many occasions to discover how much importance mechanical engineers attach to static tensile strength and to 'ductility' as measured by the elongation of a test-piece pulled to destruction, and how little Fatigue was considered by them. At first sight perhaps, it would seem to be the logical thing to do, if an axle journal became worn, to build it up to size again by arc welding ; but if such an act was permitted, that axle would almost inevitably fail in service by sudden fracture at or near the weld. Welding would have restored its static strength, but it would have ruined the metal as far as its power of withstanding alternations of stress was concerned ;

and a workshop man who thought otherwise was living in a fool's paradise. The Members would be relieved to learn that the welding of moving parts does not occur on the North-Western Railway.

At first sight it was surprising what a large adverse effect arc welding had on the Fatigue Limit of steel ; in many cases, especially if the welding was badly done, or the repair ill-designed, it might reduce the Fatigue Strength of a moving machinery member by as much as 60 per cent. or more, if corrosion was superimposed.

As Mr. Everall had shown the Members, there was adjacent to every fusion weld, a region where the parent metal had been injured by overheating during the welding process. If two pieces of steel were joined by fusion welding, and the joint was machined down to the same dimensions as the adjacent parts, the weld would probably be the weakest point under static tension. If now, the excess weld metal was not removed but was piled up at the joint so that it was the thickest part, there would obviously be a thickness at which even the weaker weld metal would be stronger than the parent metal, so that a tension failure would occur outside the weld. This may take place at almost any point, because the static strength of the overheated part was hardly affected. It was, however, enormously less resistant to shock or impact than was the uninjured metal beyond. That shock-weakness might, of course, be restored by heat treatment.

Some very interesting work had recently been done in U. S. A. on the actual F. S. of welds ; firstly, on the Fatigue failures of fillet welds, and secondly, on the F. S. of pure weld metal. The speaker said that he was taking the liberty of quoting those results because they could hardly fail to be of interest to Engineers. The speaker prefaced the figures with the remark put in for comparative purposes, that Mr. Everall's bridge steel had a tensile strength of 26 to 30 t/in², which would have meant a Fatigue Limit of about—15 t/in². The fillet welds were made by butting a piece of 1″ steel bar up against a piece of 2″ bar and welding a fillet all round. One end was loaded and the composite bar rotated until failure occurred, the weld being subject to alternating tension compression stresses of calculable magnitude. The following results were obtained :—

A.	0·30″ fillet	..8·1 t/in ².	Failed through the weld. Weld not big enough.
	0·43″ ,,	..8·3 ,, ..	Failed at base of weld, through overheated zone.
	0·85″ ,,	..7·6 ,, ..	Do. do. Note, heavier weld was weaker.
B.	Welds with 0·43″ fillets		Smooth radii machined in the weld-metal.
	Radius 1/8″	F. L. 8·4 t/in².	
	,, 1/4″	,, 8·7 ,,	
	,, 3/8″	,, 8·8 ,,	

The general rule was the sharper the 'notch' the greater the concentration of stress.

C. Fillet machined smooth, so that the section of the WM. was a 45° right angled triangle.

F. L. 6·6 t/in².

D. Leaving a slight space between the parent bars.

F. L. 5·3 t/in².

There was food for consideration there. The facts that emerged were :—

1. If the weld was reinforced so that it was above a certain limiting strength, failure occurred through the parent-metal, overheated zone.
2. Increasing the volume of the added metal lowered its F. L. (per in ². of course).
3. Machining off the rough weld surface seriously lowered the F. L.
4. In a fillet weld, if the members were not in close opposition the F. L. was very low indeed. The gap acted like an internal crack, or local stress-raiser.

Another series of tests was made on pure weld metal, the T. P. being 2″ long, 1″ dia. at centre and 1·5″ at ends, rad. of curv. 4·5″.

The following data was obtained :—

Weld as made .. F. L. 6·8 t/in².
Weld machined smooth .. F. L. 5·8 t/in².
Unmachined annealed at 540°C 7.2 (release of intl. strains).
Unmachined normalized at 925°C F. L. 5·5 t/in² (MS. lowered by grain growth).
Unmachined Peened .. F. L. 9·3 t/in² (cold worked) (smoothed).
Unmachined Annealed at 540°C .. F. L. 7·8 t/in² (removes cold-work).

These tests were made using bare wire, low carbon electrodes at 150a. The Authors stated that using a high grade flux-coated electrode they obtained a F. L. of 13·9 t/in² on a machined TP of small diameter. That figure would have been higher if the TP had been left unmachined ; and lower if the TP had been of the same large dimensions as the TP used in the tests just quoted.

The Authors stated that at a conservative estimate there was a 40 per cent. improvement in the F. L. of a weld if a coated electrode was used.

The speaker said that Mr. Everall always used coated electrodes.

Mr. K. P. Mushran said that as the Authors have pointed out in the paper, welding was making rapid strides in America, Australia and Europe, but it had still to come into its own as far as India was concerned. The presentation of that paper should help to bring home to Members the superiority of welded to rivetted joints in most forms of construction.

The North-Western Railway, as stated by the Authors in para 3. on page 80, used Quasi Arc Electrodes exclusively. The speaker hoped that the Authors would give them further information as regards the other experimental data on which that preference must undoubtedly have been based.

The speaker further remarked that Col. Macrae had referred to the feasibility of repairing Railway crossings by welding. He did not know if sufficient data on the abrasive qualities of welds was available in India and he might suggest to the Authors that they carry out a series of tests on that subject in order to determine the most suitable electrodes for that purpose. A very high manganese content would obviously be necessary for such electrodes.

In the last para. on page 87, the Authors in speaking of fillet welds, stated that the surface of contact in shear should have an area 25 per cent. in excess of that in tension. This would mean that the permissible shear stress would be 80 per cent. of the permissible tensile stress. The corresponding figures in the welding codes of Australia, America and Germany, where the workmanship was much superior to that in India, vary from 75 per cent. to 80 per cent. His own observation of certain weld specimens tested to destruction led him to the conclusion that the figure should be between 60 per cent. and 70 per cent. The Railway Board permissible stresses as quoted by the Authors on page 88 allowed a shearing stress of only 60 per cent. of the corresponding tensile stress. It would, therefore, be desirable to carry out some experiments on the shear strength of welds.

In dealing with the design of welded joints on page 89, the Authors rightly stated that in lap joints, connecting a tension member to a gusset, all concentrations of stress at the edges of the member were avoided by cutting off the corners of the flat. In that connection the speaker referred the Authors to certain test results given in a paper on "Distribution of Stresses in Welded Double—Butt Strap Joints" read by S. C. Hollister and A. S. Gelman before the American Welding Society in October 1932. Various types of strap models of the joints were prepared and observed under stress in polarised light. It appeared from those results that the most uniform distribution of stress lines, was obtained with a diamond straped strap.

The Authors have very little to say as regards the comparative costs of all-welded and all-rivetted structures, such as new plate girder or triangulated spans in India. From the examples quoted by them, it appeared that welding had an assured future as far as strengthening of existing rivetted structures was concerned, as the modifications could be carried out without interfering with or weakening the existing structure in any way during the operation.

In that connection, the speaker observed that it was the experience of Railway Engineers, and he supposed also of other Engineers, that web members of triangulated spans put into the line thirty to sixty years ago,

were insufficiently rivetted. If the rivets in such joints could be successfully suplemented by welding, it would obviate the necessity of erecting expensive staging under the spans and opening out and altering the riveted joints completely and would thus show a considerable saving in money. The speaker further remarked that he would like to know if the Authors had any information available on that point.

MR. FREAK, referring to page 93, remarked that the experimental truss purlin when tested yielded at 5½ tons or under twice the designed load of 3 tons while it failed at 7½ tons or only two and a half times the designed load which appeared unusual in face of the designed factor of safety of 4 and the speaker wished the Author to explain this.

MR. EVERALL thanked Mr. Hackney for his valuable contribution to the discussion on welding. The Metallurgist's part in the development of this Branch of Engineering was one of great importance and value.

Mr. Hackney had given some interesting remarks on the subject of fatigue of the metal in welded joints.

The Author said that he agreed with him as to its importance and particularly so in connection with railway bridge structures which were subject to considerable dynamic effects and a large range of stress.

The case, however, was not quite the same as the fatigue effects found in the metal of the moving parts of high speed engines and locomotives, the number of repetitions of loads in such cases ran into millions in a comparatively short time whereas in the case of railway bridges the repetitions of live loads might be less than one million spread over a period of 50 years.

For purposes of examining the effects of those dynamic loads on welded joints the Author said that he had made a simple testing machine in the workshop at Jhelum, in which test specimens could be subjected to alternating stresses of varying intensity. Up to the present, they have only tested specimens under comparatively high intensity of stress, because the testing under lower stresses would take too long to break the specimens.

Although the results from those tests did not give the fatigue effects under comparatively low stresses, they did show the need for a high factor of safety and they also proved that it was not sufficient to depend on the direct tensile tests upon which so much reliance was generally placed.

The Government of India Bridge Rules provided for a factor of safety of six for all shear welds done in the field and a factor of safety of four when done under more favourable conditions in a workshop.

As most bridge strengthening was done in the field, and provided that the standard of welding was sound, with such a high factor of safety as six, there was no reason why full reliance should not be placed on welded joints in railway bridges.

The use of Quasi Arc Electrodes had proved an advantage for their class of work, in so much as they could be fused at a lower temperature and with a lower amperage than the M. W. P. Electrodes. The avoidance of unnecessary and undue distortions due to temperature effects in long bridge members and girders was one of great importance and they had found that the Quasi Arc Electrodes with blue asbestos covering answered the purpose better than electrodes of other makes which had been tried out by them, although the actual strength per square inch of the Quasi Arc Welded joints might be less.

Another question raised by Mr. Mushran concerned the use of welded joints in combination with rivets or bolts for tension or compression members. The Author said that their knowledge of the distribution of the stresses in that connection was not very great at present. Some authorities stated that the rivetted joints were presumed to take only the dead load and at most one-third of the live load. The question of distribution was complicated by the slip and bending which took place in the rivets.

The Bridge Standards Committee of the Railway Board at their last meeting had asked the North-Western Railway to carry out some investigations in that connection with a view to determining the proportion of loads the rivets and welds would each take. Those investigations were now being made and it was hoped that some reliable data would shortly be forthcoming on the subject.

It was thought that possibly a reasonable distribution of the stresses could be obtained by keeping the welded joints behind the group of rivets so that part of the load was first transferred through the rivets.

Plate XIII, Fig. (1) below illustrated the proposed arrangement of a rivet-cum-weld joint.

To verify the results, the writer proposed to take careful measurements on a series of test specimens in a testing machine designed as shewn on Fig. 2.

The specimen, which was shewn in position, had six rivets connecting two plates together and this would be subjected to loads equivalent to 3 tons per square inch shear stress in the rivets. With the joint under this pre-stressed condition, it was proposed to make the welded joint as shown and then apply further increments of loads to the specimen by means of the calibrated jacks. Stress measurements would be taken at different stages by means of a number of Huggenberger Tensometer instruments applied at Stations A. and B. ; the measurements at "A" being an average of the total load applied to the bar and the measurements at "B" the residue of stresses passing from the group of rivets to the welded joint.

It was proposed to test out specimens built up with 3/4," 7/8" and 1" dia. rivets and with plates varying from 3/8" to 3/4" in thickness.

A successful analysis of this problem would enable bridge engineers o apply the method of rivet-cum-weld connections with greater confidence o the strengthening of bridges, especially in the case of members already :arrying considerable dead load stress, and in determining the proportion of the stresses carried by the rivets and the welded joint.

Colonel Macrae had mentioned the possible use of welding in con- nection with Permanent Way. There appeared to be a large field for useful work available in connection with the reconditioning of worn points and crossings. Wear took place over restricted areas and it was possible to build up the metal on the worn sections by welding on suitable elctrodes. Electrodes were specially made for the purpose of reinforcing carbon steel rails as well as manganese steel. It was possi- ble to repair and recondition a worn crossing in position for a matter of from Rs. 60 to 75, and as a new crossing, for 90 lbs. rail track, cost something like Rs. 400, the savings that could be effected were consi- derable. Special Diesel Engine welding sets were now available for that purpose and were provided with Auxiliary tools such as Electric grinders and drills. The plant could be considered as a dual purpose machine and could be used for bridge repairs as well as for track work. It could be placed on the ground alongside the work or mounted for working in a railway wagon.

The practice of reconditioning crossings by welding *in situ* had been in use for some years on the London and North-Western Railway in England and an excellent job was made of such welding at a cost of about Rs. 40 for each crossing.

In certain countries in Europe quite a lot had been done in the weld- ng of the running rails together and the elimination of the bolted rail oint, the elimination of such rail joints was considered important on bridges in Switzerland as it reduced a big percentage of the impact on small bridges.

Mr. Berridge wished to thank those Members who had so kindly oined in the discussion on the paper.

Colonel Macrae had mentioned that Metallic Arc Welding had been tried out on track work on the North-Western Railway. The speaker would like to mention one instance which came to his notice, where welding had been applied to a rail joint on the Indus Bridge at Kotri. Members of Congress would be familiar with the special reinforced concrete abutment about which Mr. Everall had spoken to them in a previous paper. During the strengthening of this abutment at the Kotri end of the Indus bridge, every effort had been made to avoid impact stresses on the badly cracked masonry below. The rail joints in the crack had been welded and although proper manganese steel electrodes had not been used, impact due to the hammering of the wheels passing over these joints had been eliminated.

Mr. Hackney had mentioned that the tensile strength of bridge steel was 26 tons per square inch but the speaker was of the opinion that

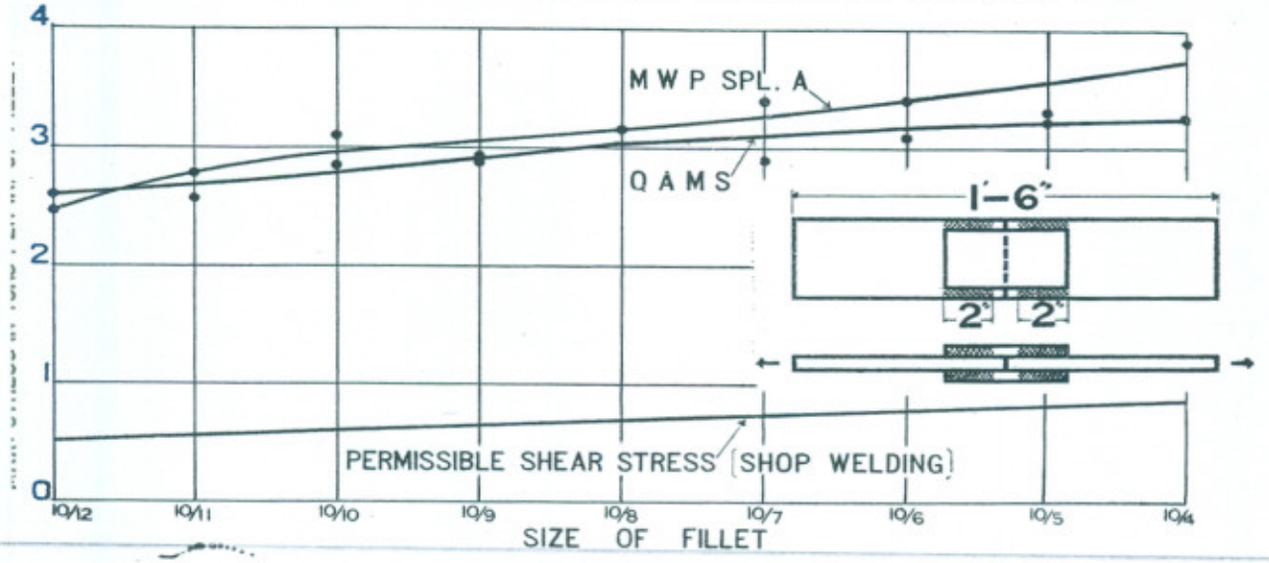
Mr. Hackney must have been referring to older qualities of steel as th steel used in the Bridge Department of the North-Western Railway had maximum stress of 28—32 tons per square inch.

Mr. Mushran had raised certain points regarding the distributio of stress across a fillet joint. Mr. Mushran had indicated how the inten sity of stress at the corner of the weld might be avoided and the speake agreed with Mr. Mushran's suggestion.

Mr. Freak had pointed out that the factor of safety of the experi mental all-welded truss purlins was only a little more than two. In repl the speaker observed that Mr. Freak's remarks appeared to be correc but he considered it only fair to state that these truss purlins were no designed by the Bridge Department of the North-Western Railway.

PLATE Nº XI

STRENGTH OF LONGITUDINAL FILLETS



STRENGTH OF TRANSVERSE FILLETS

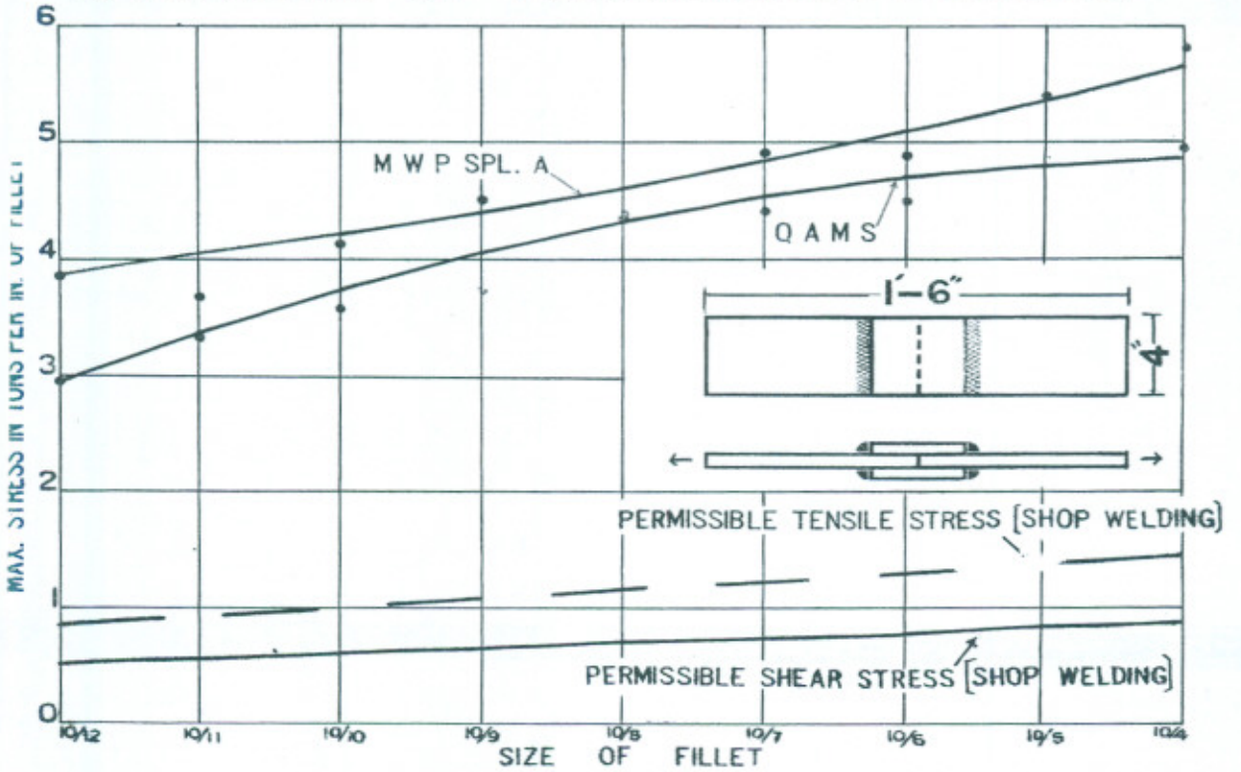
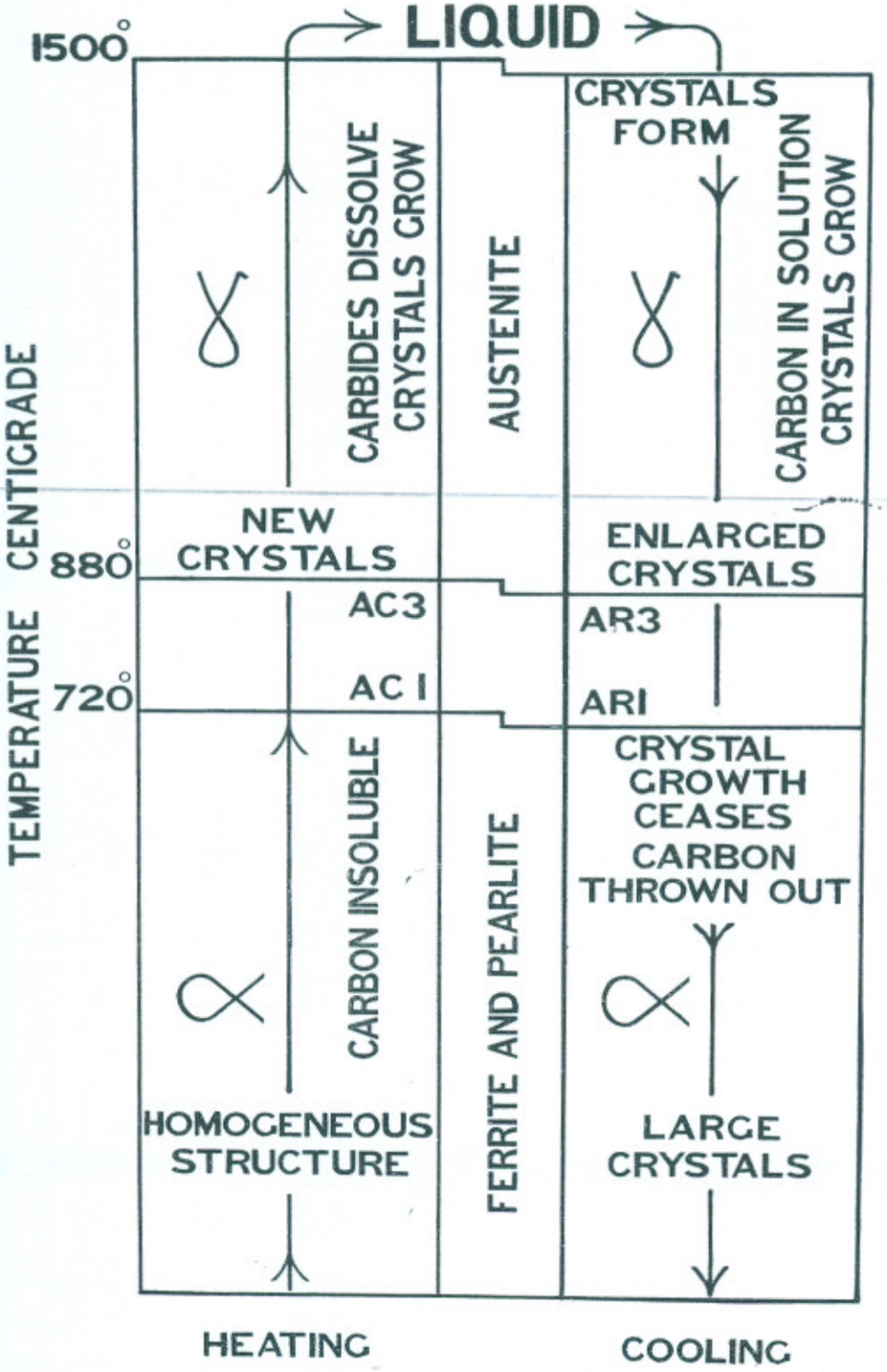


PLATE No. XII

CHANGES IN STRUCTURAL STEEL (0·2 % CARBON) WHEN HEATED



MACHINE FOR EXAMINING THE STRENGTH OF RIVET-CUM-WELD JOINTS

PLATE XIII

FIG 1

RIVET-CUM-WELD TEST SPECIMEN

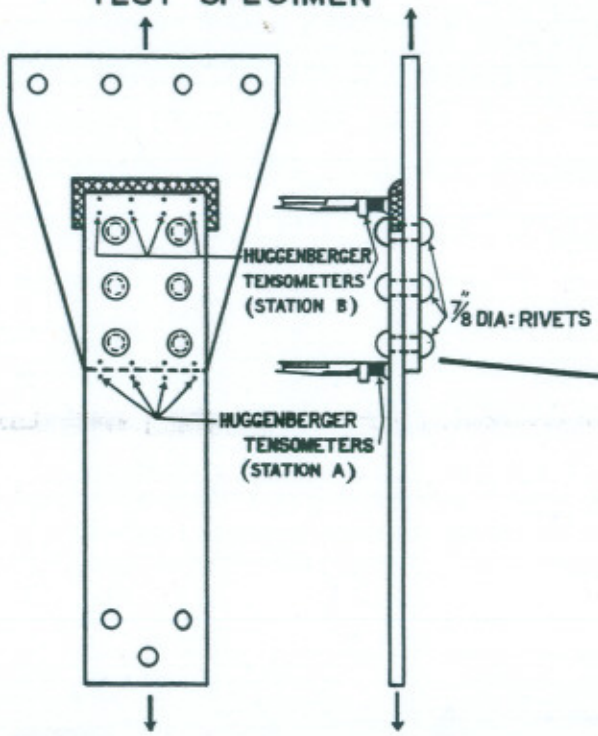
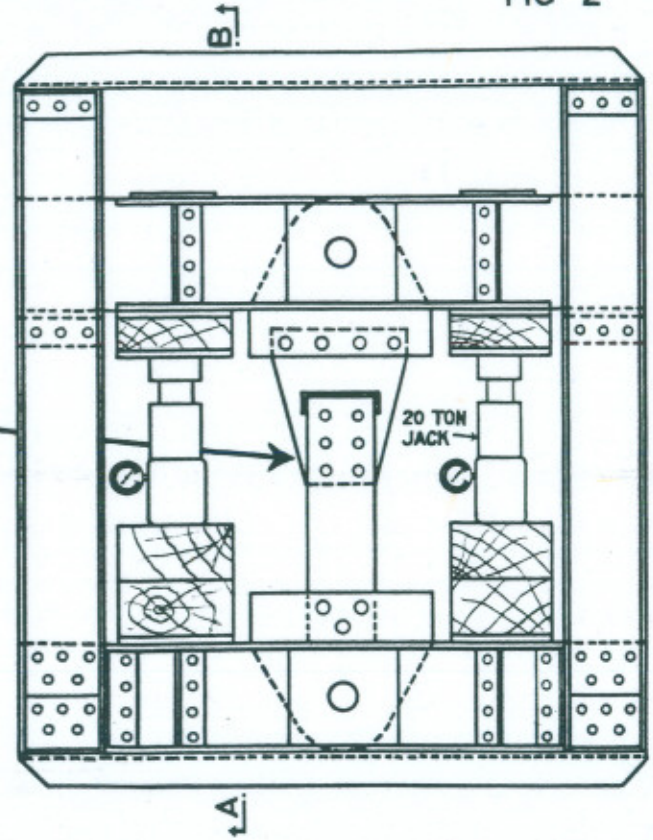


FIG 2



SECTION A B

