PAPER No. 160.

DESCRIPTION OF SEVERAL UNUSUAL STRUCTURES ADOPTED IN BRIDGE CONSTRUCTION.

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The writer in choosing this subject for his Paper thought that members might be interested to have an account of some features of bridgework undertaken by the Bridge Branch of the North-Western Railway, which are somewhat outside usual Engineering practice and which were introduced largely on account of the special requirements which governed for each bridge. Four such works are described :—

I. The reinforcement of a cracked abutment of the approach span of the combined road and rail bridge across the Indus at Kotri.

II. Method of reinforcing the Tilted well of pier No. 4 in the West Channel of the combined rail and road bridge across the Chenab at Chiniot.

III. A new form of portable steel road bridge made of interchangeable units as used for constructing bridges on the Chitral Road and other roads in the N.-W. F. Province.

IV. Repairs to the Kohala Bridge across the Jhelum on the Rawalpindi-Kashmir Road.

The writer wishes to thank the N.-W. Railway Administration for permission to present this Paper.

No. I.

The reinforcement of a cracked abutment on the approach span at the Kotri end of the Kotri Bridge across the Indus.

This abutment carries the shore end of the 100 ft. end span and is of the return wall type. A general arrangement of the existing structure is shown in half plan in dotted lines to the right of Fig. 1. The original foundations were carried down to 13·0 feet below H. F. L. as may be seen in the sketch on page (3). During the flood season the ground becomes water-logged at this point. Probably owing to insufficient support from the ground at the upstream and downstream ends of the foundations, a vertical crack had developed, over one inch wide at the bedstone level, and extending to the bottom of the foundation where it tapered to less than half an inch.

Below the bottom of the foundations, sand exhibiting the qualities of quicksand is found.

Owing to the elastic nature of the soil, the old abutment is liable to settle further on imposing heavier loads and there is a likelihood of

the two portions of the abutment on either side of the crack spreading further apart or of tilting away from each other.

The condition of the masonry and the concrete (brick ballast in lime) is generally good, except that the outer face of the concrete has deteriorated to a depth of some 6".

The abutment is founded on very poor soil, *i.e.*, sand and silt, which, when worked, becomes elastic.

When considering methods for strengthening the abutment, the following conditions had to be kept in view :—

(*a*) That the pressure on the foundation should be kept down to a low figure ;

(*b*) That, as far as practicable, the water bearing soil should be confined under the foundations ;

(*c*) That the design should embody some means of stabilising and preventing the old abutment from splitting further apart, and of relieving the abutment of the whole weight of the 100′ approach span carrying the new cantilever roadways and the heavier railway loading ;

(*d*) That the work should be capable of being executed under railway traffic.

To cover these conditions an unusual design of reinforced concrete structure has been constructed. This is illustrated by Figs. 1 and 2 and the left half of Fig. 3 on Plate I.

Being of cellular construction, the structure imposes the minimum of added dead load to the foundations.

It has the advantage of avoiding the cost of a completely rebuilt abutment, using a raft foundation or reinforced concrete piles, which were considered and rejected chiefly on the grounds of cost and the difficulties of carrying out the work under traffic.

The reinforced concrete work was constructed clear at the sides and at the top of the original brickwork, after which, the full dead and present live loads were taken by the new work alone. Having allowed sufficient time for shrinkage and settlement, the clear spaces between the reinforced concrete and the masonry were filled with cement grout injected under pressure.

Some of the more important calculations are overleaf.

Main Cross Beam and Slab.

This was initially assumed to be a simply supported beam of 27 ft. effective span (*see* Fig. 3) resting on two vertical supports. Actually the reactions from the supports are inclined towards the centre line of the bridge but sufficient stiffness has been provided at the ends to counteract the tendency for the supports to spread apart.

The total reaction from each of four end bearings resting on the abutment, including dead load of girders and maximum live load end shear under present train loads=97 tons. (*See* Fig. 3.)

$\therefore$ BM due to total superimposed load

$$= \left\{ \frac{194 \times 27}{2} - \frac{194 \times 21{\cdot}5}{2} \right\} \times 12 \times 2240$$

$$= 14{,}350{,}000 \text{ inch lbs.}$$

Dead Load of main beam and slab=55·8 tons

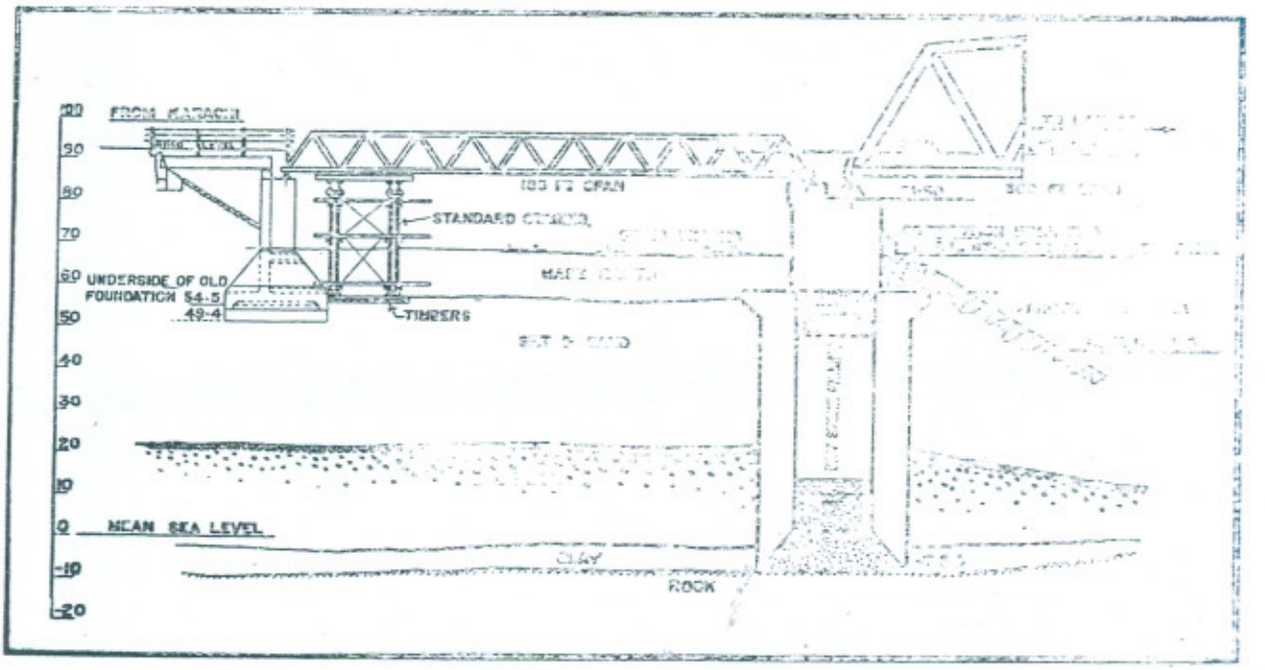
$$\text{BM due to this} = \frac{55{\cdot}8 \times 27 \times 12 \times 2240}{8}$$

$$= 5{,}080{,}000 \text{ inch lbs.}$$

Reverse BM due to cantilever extensions = 800,000 inch lbs.
Net BM due to DL of reinforced concrete=4,280,000 inch lbs.
Total BM to be resisted =18,630,000 inch lbs.
Resisting moment of slab =20,000,000 inch lbs.

It will be seen that the main slab 3′ deep which takes the span reactions is strengthened in bending by a beam 8 feet in total depth and 2ft: wide, incorporated at the back of the slab which acts as a retainer for the fill under the formation and as a ballast wall. At sections such as CC Fig. 3 between the bridge bearings and the RC supports, the RC beam is subjected to torsional stresses in addition to bending and vertical shear because the line of application of the load from the superstructure, is eccentric to the centroid of the main beam. (*See* Fig. 2.)

A diagram of the foundation loading under the most severe combination of dead and live loads and horizontal forces is shown below Fig 2.



The area of that portion of the original foundation retained after remodelling has been included in calculating the foundation pressure.

ROADWAY CANTILEVERS. (*See* Fig. 3).

Dead load (Cantilever) ..	..1·8	tons
Road concrete ,, ..	..3·7	tons
Girders ..	..2·0	tons
Live Load plus Impact ..	..9·5	tons
Total ..	17·0	tons

BM=17×2240×72=2,740,000 inch lbs.
Effective depth=57″
Using 6—7/8″ dia. rods

$$\text{Resisting moment}=\frac{95\times 24\times 57\times 57\times 6\times \cdot 601\times 100}{\cdot 675\times 57\times 24}$$

=2,900,000 inch lbs.

Execution of the work.

The abutment end of the existing 100 ft. approach span was supported from the ground on standard interchangeable staging (*see* diagram on page 3) to relieve the original masonry of the weight of the superstructure and the live load. The original bedstones were removed and also the rounded ends of the abutment, and the excavation was carried down for the new concrete raft during the last working season.

Unfortunately it was not possible to complete the raft foundation before the river level rose and the work was, therefore, suspended after being well protected by sheet piling during the last rains. Work was recommenced last October.

Difficulty was experienced in excavating for the foundations of the R.C. raft owing to the nature of the soil, and much trouble occurred in the excavation of the footing walls, where in spite of the fall of the river below the foundation level, water was encountered. Pumping was resorted to, but the level of water in the excavation was reduced with great difficulty owing to the very heavy influx of sand as pumping continued. Shoring was constantly disturbed and fell in owing to this influx of sand which was only finally overcome by excavating short lengths of foundation at a time and lining the excavation with a layer of cement concrete to the shape of the foundation required.

In order to ensure that the new reinforced concrete work should be entirely clear of the old masonry during construction, a layer of sand 2″ thick was laid along the top of the old masonry which supported the shuttering. In order to remove the shuttering after the concrete was cured the sand carpet was blown out by compressed air and the boarding withdrawn. In the case of the vertical joints, 2″ of mud plaster was applied to the old masonry at the abutting faces. This plaster was removed by water jet before grouting.

Sections of the reinforcement were built up in the bridge workshop and placed in position at the site in large units. A portion of the reinforcement is illustrated in Photo No. 1. The concrete was mechanically mixed and placed in position by means of suitable chutes. Photo (No. 2) shows the completed structure.

No. II.

Method of reinforcing tilted well of pier No. 4 of the combined road and rail bridge across the West Channel of Chenab River at Chiniot.

During the progress of work for constructing this bridge, certain problems arose in connection with the tilting of the well of pier No. 4 in the West Channel.

The contractor commenced work on the wells in October 1928, and pitched curbs for wells Nos. 3 and 4 and built the first 12 ft. of masonry. Before sinking was put in hand, a five-feet flood came down on the 2nd December and caused the wells to tilt. Considerable scour took place at well No. 4 and after the flood had subsided, only the downstream end of the well was visible. Nothing further was done at this well until the 24th February when an island was constructed and the well carried down to 94 ft. below L. W. L. At this point it was found that there was a longitudinal tilt amounting to 8′-0″ and a cross tilt of 3′-6″.

It was decided by the Chief Engineer in November 1929, that further sinking would be unwise as the tilt would certainly become worse. Calculations showed that the well would be stable and not overstressed if a pier were built eccentrically above the well cap, provided that special reinforcement was arranged to limit the extreme intensity of compression stress in the well brick-work immediately under the eccentric pier.

The position for the location of the end bearings of the girder was arranged to keep the bending stresses down to a safe value. This necessitated lengthening one bay of the 205′-7″ span between wells 3 and 4 by 6′-1″ making the length of the span 211′-8″, while the girders intended for the next span which is the Western approach span were accommodated by locating the abutment further back.

It might be explained here that the well of pier No. 4 was in the shape of a double octagon with two octagonal dredge holes 9 feet diameter with centres 16 feet apart.

The Bridge Branch was asked to prepare the design of the special pier reinforcement and also to undertake the work at site.

A general arrangement of the reinforcement is shown in Figure 1 Plate II.

Without such provision the calculated intensity of pressure in the brick-work under the toe of the trestle would have exceeded 9 tons per square foot. With the reinforcement, this has been reduced to a maximum of 5·5 tons/sq. ft.

In the design, the reinforced concrete well cap was assumed to take the dead load of the superstructure which is erected upon it. This condition was assured by under-cutting the well brick-work under the toe of the cap to enable the reinforced concrete cantilever to take stress and deflect under the weight of the girders and the trestle forming a skeleton frame-work for the eccentric pier which was completed later. This is illustrated by Photo No. 1.

After erection of the trestle and girders and the completion of the road-metalling, the under cut brick well was built up to the underside of the well cap and finally completed by pressure grouting. The new pier surrounding the steel trestle was then built to complete the work at well No. 4.

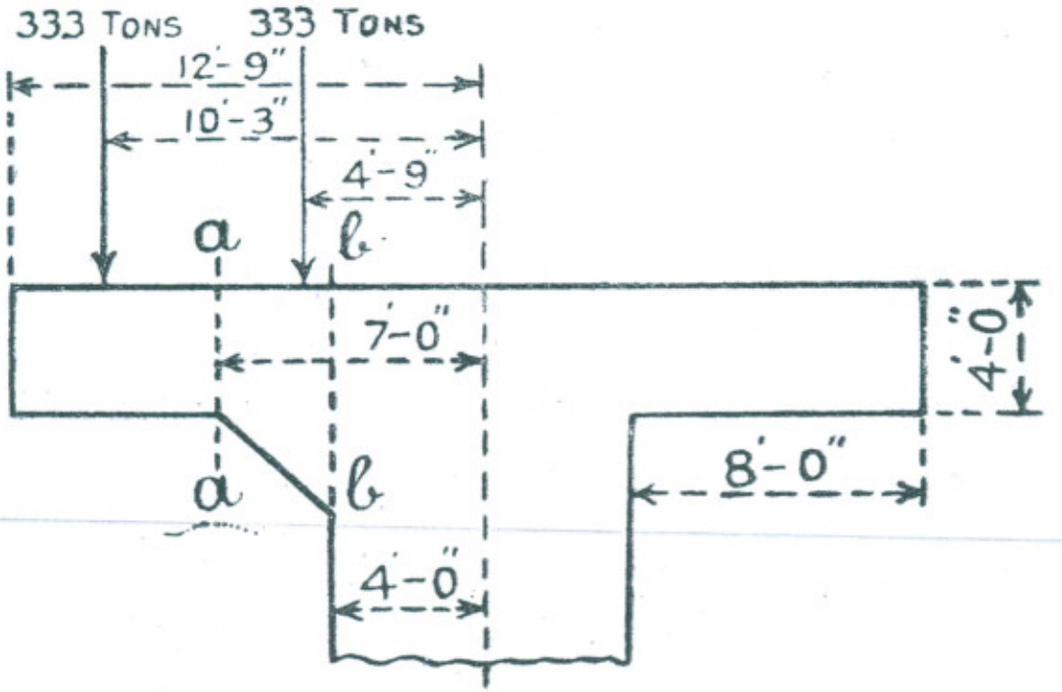
The loads which had to be dealt with are as follows :—

Dead load girder reactions	440 tons
Road metal	140 ,,
Trestle	45 ,,
Reinforced concrete cantilevers	120 ,,
Total superimposed load carried by the well reinforcement.	745 ,,
Railway live plus impact loads	575 ,,
Roadway live load	115 ,,
Total live load	690 ,,
Dead load of pier	500 ,,
Total load assumed taken by brick-work ..	1,190 ,,
Total wind load on leeward trestle leg with both spans loaded.	150 ,,

As this figure is only 8% of the total dead and live loads on the pier, it was neglected in the calculations for reinforcement.

Calculations for Design.

Total load through trestle base taken by reinforced concrete cantilever.



CROSS SECTION OF WELL CAP THROUGH CENTRE LINE OF DREDGING HOLE

To determine goose neck reinforcement.

It is assumed that the whole of the concrete load to the left of section a a is taken by the 8 ft. width of each goose neck only.

Mean length of slab at section a a=36 ft.

$$\text{On each k, weight} = \frac{5{\cdot}75 \times 2 \times 36 \times 150}{2240}$$

$$= 27{\cdot}7 \text{ tons.}$$

B. M.=268,000 inch lbs. per foot of width

Steel moment=333×39×140=1,820,000 inch lbs. per ft. of width

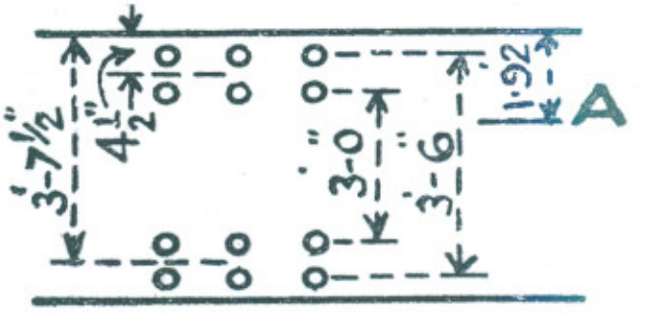
∴ Total moment=2,088,000 inch lbs. per ft. of width.

Area of $1\frac{1}{4}$" bar = 1·22 sq. in.

Area of tension steel per ft.

$$=\frac{1\cdot22\times2\times12}{5}$$

= 5·9 sq. in. =1·13%.



$\frac{d^1}{d}$ = ·10, p = p′, k = ·335, j = ·89.

Moment of resistance=16000×43·5×·89×5·9

=3,650,000 lbs. with two rows top and bottom

Shear.

Steel .. 333 tons

Concrete=2×27·7 .. 55·4 ,,

388·4 tons total.

$$=\frac{388\cdot4}{16}=24\cdot3 \text{ tons/ft. of width.}$$

Shear per linear inch per ft. width $=\frac{24\cdot3\times2240}{\cdot89\times43\cdot5}$

=1410 lbs.

With 6" spacing of stirrups and stress of 10,000 lbs/sq. in.

Area required $=\frac{1410\times6}{10000}$

= ·845 sq. in.

=$\frac{3}{4}$ bars @ 5" Crs.=1·05".

Area provided =1·43 sq. in.

Bond stress $=\frac{1410\times5}{1\cdot25\times\pi\times2\times12}$ = 74·6 lbs./sq. in.

Max. permissible=80 lbs/sq. in.

On Section b b.

Shear =388·4+335+25

=749 tons total.

=46·7 tons per ft. of width.

Shear per linear inch per ft. of width.

$$= \frac{46{\cdot}7 \times 2240}{72 \times {\cdot}88}$$

$$= 1652 \text{ lbs.}$$

With 6″ spacing of stirrups

$$\text{area required} = \frac{1652 \times 6}{10000}$$

$$= {\cdot}991 \text{ sq. inch per ft. of width.}$$

$\frac{3}{4}$ bars @ 5″ Crs. $= 1{\cdot}05$ sq. inches per ft. of width.

$$\text{Bond stress} = \frac{1652 \times 5}{1{\cdot}25 \times \pi \times 3 \times 12} = 58{\cdot}4 \text{ lbs./sq. inch.}$$

B. M. on Section b b

Concrete moment per foot of width.

$$\frac{27{\cdot}7 \times 5{\cdot}875 \times 2240 \times 12}{6}$$

$$= 730{,}000 \text{ inch lbs.}$$

Steel moment per ft. of width.

$$\frac{333 \times 75 \times 2240}{2 \times 6} + \frac{333 \times 9 \times 2240}{2 \times 6}$$

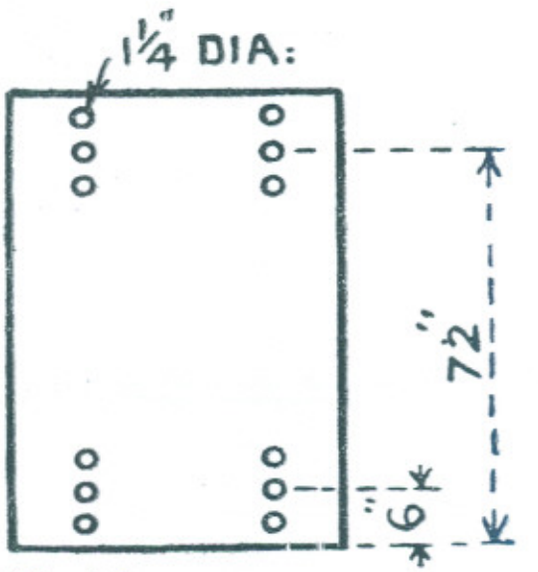
$$= 5{,}220{,}000 \text{ inch lbs.}$$

Total moment $= 5{,}950{,}000$ inch lbs.

Area per ft. run $\frac{1{\cdot}22 \times 3 \times 12}{5}$

$= 8{\cdot}8$ Sq. in.

$= 1{\cdot}02\%$



$\frac{d^1}{d} = {\cdot}08$, $p = p^1$

$k = {\cdot}345$, $j = {\cdot}90$

Moment of resistance $= 16000 \times 72 \times {\cdot}90 \times 8{\cdot}8$.

$= 9{,}100{,}000$ inch lbs.

Stress in brick-work under Well Cap.

Load taken directly by brick-work.

Live load railway	..	575 tons
Live load road	..	115 ,,
Dead load pier	..	500 ,,
Total	..	1190 tons

Effective area of brick-work underneath.

Well cap=649·6 sq. ft.

Moment of inertia=28423 ft.^{4}

Distance of neutral axis from extreme " fibre "=12·5 ft.

$$\text{Direct thrust} = \frac{1190}{646{\cdot}6} = 1{\cdot}835 \text{ tons/sq. ft.}$$

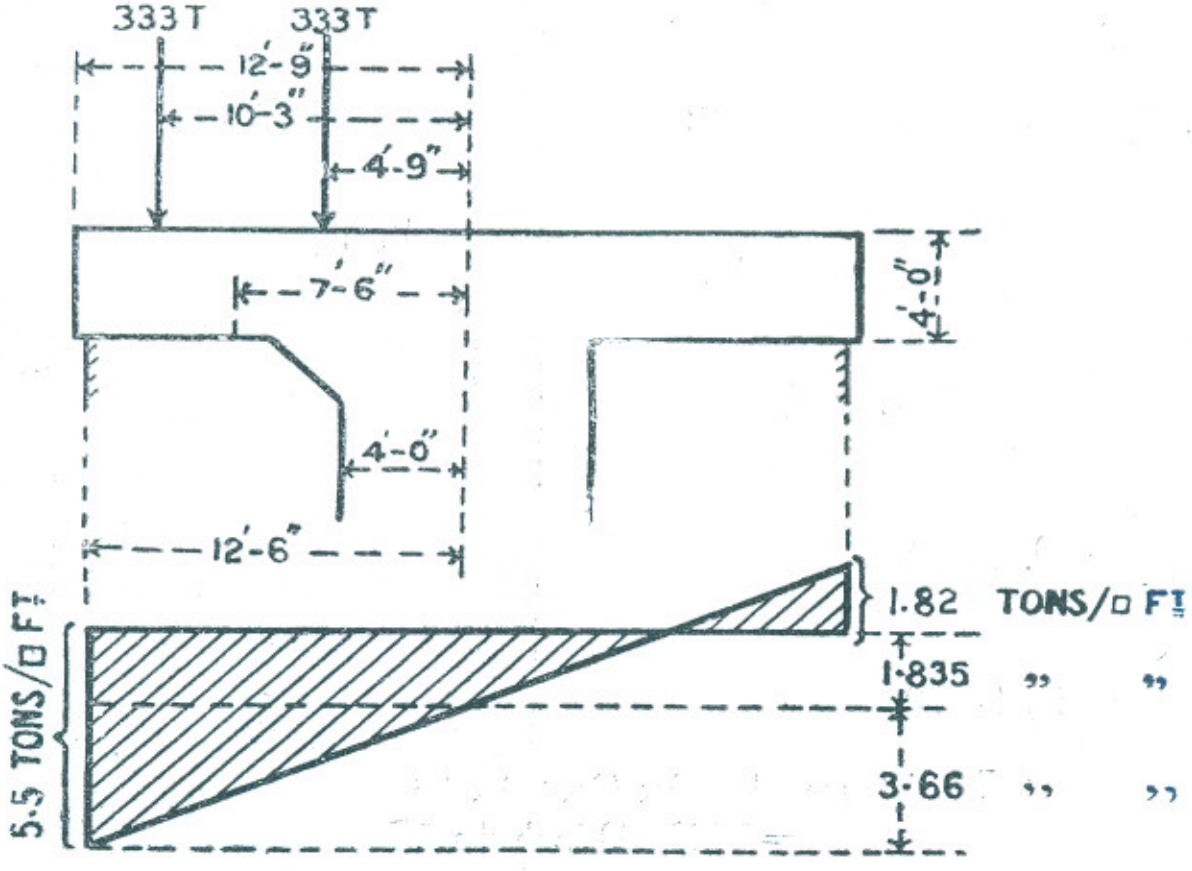
Eccentricity GG of applied loads = 7'-0"

$$\text{Bending stress} = \pm \frac{1190 \times 7 \times 12{\cdot}5}{28423}$$

$$= 3{\cdot}66 \text{ tons/sq. ft.}$$

∴ Maximum compressive stress =1·835+3·65.
=5·5 tons/sq. ft.

,, tension ,, =3·66—1·835
=1·82 tons/sq. ft.



Execution of work.

No great difficulty was experienced in the execution of the work after the dredge holes had been dewatered although much thought had to be given to the fixing of the well reinforcement. The vertical rods were 1¼" dia. and consequently heavy and difficult to place in the confined space of the dredge holes. After fixing, concrete was poured in up to within six feet of the top of the wells and then stopped to allow the shuttering and reinforcement for the pier cap to be fixed. The arrangement of reinforcing rods in place is shown on Photo No. 2.

Finally, the cap was cast and the work completed.

For concreting two half yard rotary concrete mixers mounted on country boats were used, one mixer to each dredge hole. These deposited concrete continuously by means of chutes and flap bottom skips.

In Figure I it will be noticed that a number of rolled steel joists in pairs are shown placed transversely in the dredge holes. These were provided to act as keys between the central mass of concrete and the masonry of the wells.

Testing.

At the time of the tests the staging and camber jacks had not been dismantled. It was therefore, possible readily to apply and remove the dead load reactions taken by the trestle and thus provide a test load.

This applied load was 430 tons and amounted to 53% of the total designed load taken by the reinforcement.

The positions of the instruments during the tests are shown in Fig. I., Plate II.

A Deflectometer was placed under the cantilever nose, Fereday Stress recorders were attached to the goose neck at F and Huggenberger extensometers at H in Fig. I.

The maximum deflection of the cantilever noses in relation to the well top under 430 tons was 0·011 inch. The permanent set determined by means of a precision micrometer deflectometer was zero on removal of the test load. It was not convenient to attach extensometers at the points of maximum calculated stress; however, Huggenberger extensometers attached to the tension steel at H in Fig. I. indicated a stress of —6,180 lbs. per square inch under 430 tons. This corresponds to a stress of —6,850 lbs. per square inch calculated at this point.

Stress readings considerably less than those calculated by usual assumptions were recorded by the Fereday optical recorders attached at F in Fig. 2.

In conclusion, it might be mentioned, that the scheme has proved successful.

No III.

Standard Portable Steel Road Bridges.

Some time ago the writer was asked by the Deputy Chief Engineer, Military Works, N.-W. F. P., Colonel (now Brigadier) C. H. Haswell, Chief Engineer, Western Command, for the design of a light type girder bridge capable of carrying mechanical transport, the girder units to be made light enough for transport on pack mules or camels. These structures, one of 112′ and one of 72′, were required on the Chitral Road, which hitherto had only been used for pack transport, to replace bridges washed away by the floods of August 1929 and which could only be reached by pack animals over mountainous tracks, the latter so difficult that 13 miles represented an average stage for a day and the cost of transporting bridging materials from Durgai to Dark is so excessive that special consideration had to be given to the design of the girder-work to make it as light and as portable as possible, and at the same time contain the minimum number of different parts. It was also necessary to arrange for the girders to be erected with the minimum amount of tackle and equipment.

The designs, as prepared by the writer, and fabricated and used for bridges on the Chitral Road and other parts of the Frontier, are illustrated by the figures on plates Nos. 3, 4 and 5 and photographs Nos. 1 and 2. These designs have, however, been adapted so that a much greater range of use can now be made of this type of girder. As designed it forms what is in effect a portable Mecanno Road bridge span and provides for an improved form of constructing and of erecting bridges.

The types are particularly applicable to bridges for Military purposes or for opening up wild and inaccessible country where the bridges are difficult to reach, for example in inaccessible positions which have no railways or roads communicating with the site.

The spans are comparatively simple to assemble and erect and ordinary intelligent labour with the assistance of a few skilled men can rapidly construct these bridges without difficulty.

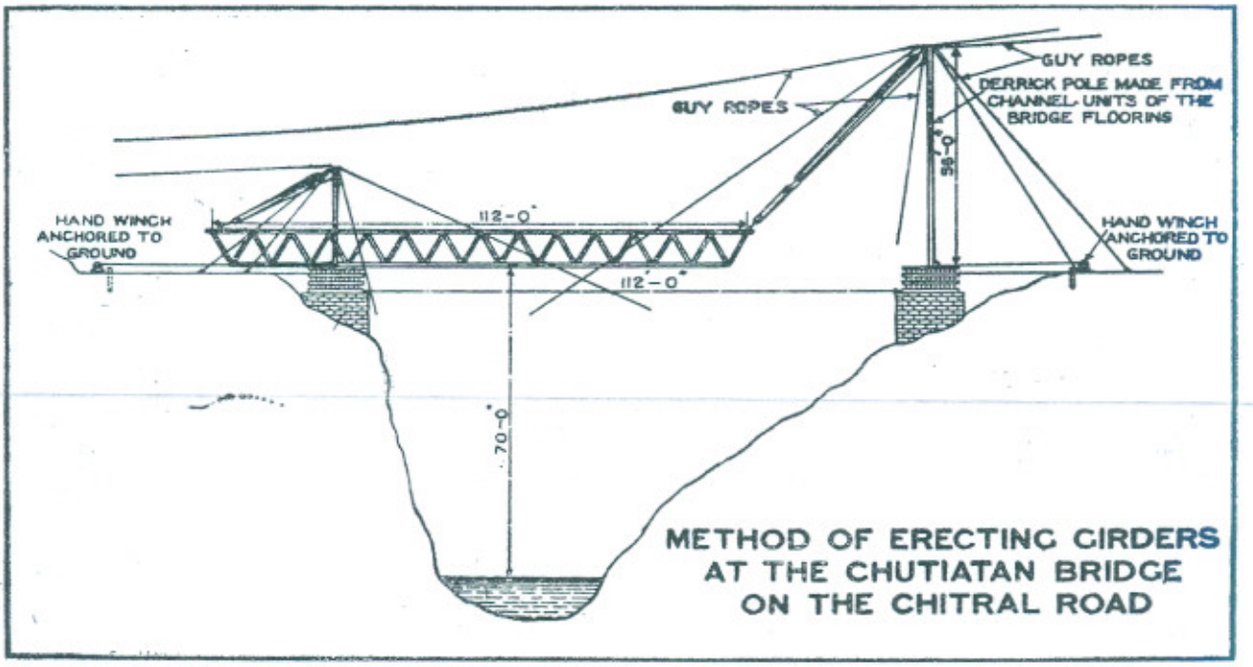
The members, when assembled, form very stiff sections and as such, are specially suited for girder erection. They can, when fully assembled, be launched or cantilevered out for a considerable distance or lifted by means of a single derrick without distortion taking place.

These are factors of great importance in the erection of bridges at sites where space and other facilities are limited.

The Chutiatan Bridge on the Chitral Road is an example illustrating the possibilities of erecting these girders in a difficult position. The bridge spans an opening of 112′ at a height of 70 feet above water-level.

A girder was first assembled complete on one side of the Nullah and launched out a distance of 50′ beyond the abutment, the girder end was then attached to the lifting tackle of a second derrick pole located on the opposite bank and the girder lifted and moved forward by means of winches to its correct position.

The diagram below and photo No. 1 illustrate how the work was carried out.



The derrick pole 56′ in height as may be seen in the sketch was made from the cross girders of the bridge, and used temporarily for the purpose.

A small bridge gang, consisting of 10 ordinary Punjabi Khallasies of the N.-W. Railway Bridge Branch assisted by 30 untrained local Pathans carried out the work in quick time and without difficulty.

The girders are made from ordinary standard commercial sections easily obtainable in India, and when fabricated by mass production methods, the cost per ton is not greater than that of ordinary bridge work of first class quality.

Girders of this type, if kept in stock, would enable arrangements to be made in advance for bridges for a hill-road urgently required where no detailed survey was available, by estimating the total length of girders required from a contour map or aerial survey. Another use for this type of girder would be to replace bridges destroyed by floods or by other causes and to provide for the rapid opening up of communications.

Loads.

The girders are designed to carry the loads as shown on plate No. 4.

Extended use for carrying heavy single unit loads.

The use of these bridges can be still further extended for carrying a single heavy unit, weighing upto 20 tons, by duplicating the members forming the cross girders and strengthening the decking.

Constructional features.

A standard width of roadway of 12′-0″ between wheel guards is provided on all spans.

The span can be varied within wide limits, the panel lengths are each 8 ft. and spans from 40 ft. to 128 ft. can readily be constructed and used for building either through or deck type bridges. The table shown on plate No. 6 indicates to some extent a few of the spans that can be made and also the number of units and their weights in each span.

The same standardized unit, consisting of a simple channel 8″×3″×7′—11$\frac{7}{8}$″ long, with all its holes drilled from steel jig templates, is used for both the boom and web members and also for the cross girders. The members are carefully check gauged after fabrication to ensure their interchangeability. The same method of construction is applied to other component parts such as the gussets, covers, batten plates and lateral bracings.

In the longer spans of 88′ and above, the boom sections towards the centre of the span are reinforced by doubling the channel unit and placing one on either side of the gusset plate, the same method is also used for reinforcing the web members as the ends of the longer spans.

Adequate bracings of standardized type are provided in both the deck and through spans.

The details of the above features are illustrated on plates Nos. 1 and 2.

Three-quarter inch diameter holes are used for all connections throughout the structure.

If a more permanent form or structure is required, the bolts can be replaced by rivets.

For the road decking, deodar or other timbers of equal strength, 6″ thick, bolted to the cross girders may be used, or as an alternative, light steel troughing 4″ deep×$\frac{1}{4}$″ thick, filled with concrete.

For spans 80′ and below, bearings made of flat plates are provided, for spans of greater length a built up steel rocker bearing of the type shown on plate No. 5 is used. This provides for adequate means of expansion and contraction in the bridge.

This type of bearing is used for both the fixed and free ends of the spans, the only difference being that in the fixed end bearings, shim packings are inserted between the inner faces of the lower frame and the link.

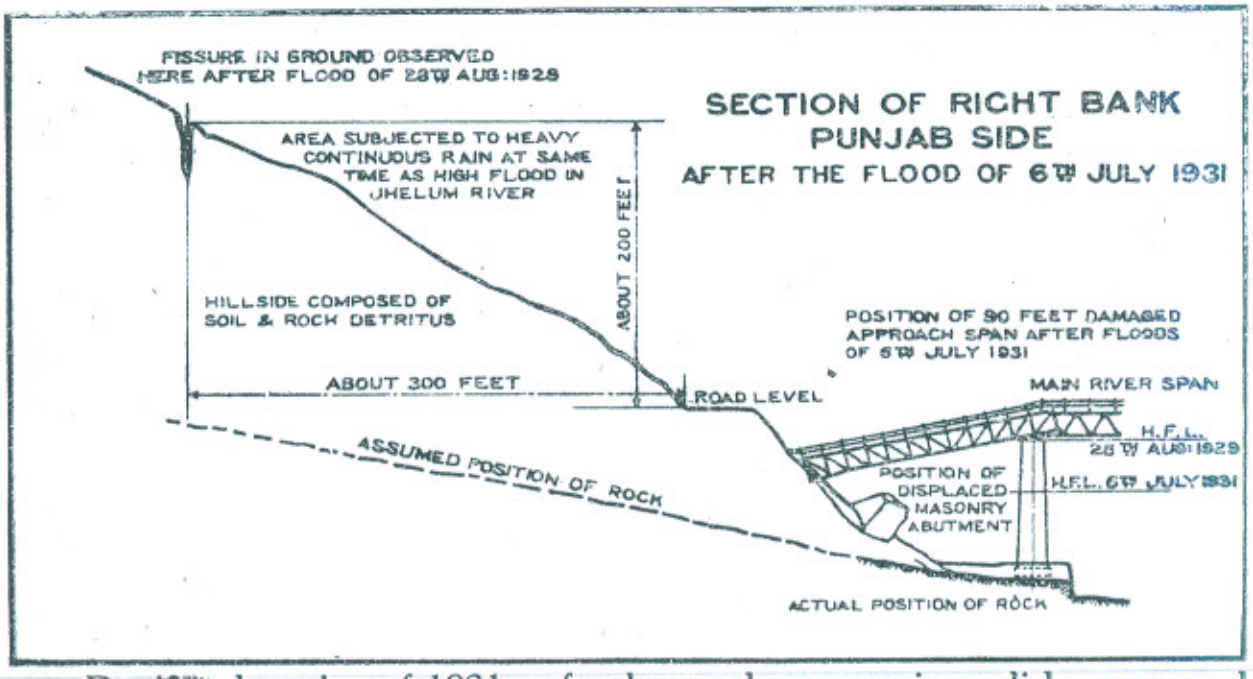
It is possible with this bearing to locate the loads definitely through the centre of its support. Movement is obtained by means of the pins and links which form an integral part of the structure and as such can be bolted to the pier, trestle or pile.

No. IV.

Reconstruction of the Kohala Bridge across the Jhelum on the Rawalpindi-Kashmir Road.

A suspension bridge originally carried the Grand Trunk Road across the Jhelum River at Kohala but this was swept away during the flood of 1893 and was replaced by a three span girder bridge on a new alignment about 240 yards upstream. This bridge was composed of one main 130 ft. span with a 90 ft. approach span at either end. On account of the unstable condition of the hillside on the Punjab side there was a tendency for the girders to be pushed longitudinally over the supports towards the left bank and prior to 1929 it had been recorded that the movement of the girders had been sufficient to cause the Kashmir approach span to become jammed up hard against its abutment. However, the bridge remained in service until August, 1929, when, as a result of continuous rain and high floods a considerable landslide occurred at the Punjab side abutment which caused severe distortion in the booms of of the 90 ft. approach span on that side. The upstream bottom boom of the main river span was also damaged by floating debris. Damage to the 90 ft. span was so severe that it was found necessary to renew the main girders. This was done by the North-Western Bridge Department in 1929 who also made the necessary adjustments at the abutment to compensate for the movement which had taken place. The writer, early in September, 1929, just after the flood, examined the hillside at the Punjab end of the bridge and found a long fissure situated at about 300

ft. from the back of the abutment and about 200 feet above the floor level of the bridge. This fault is indicated in the diagram.



During the rains of 1931, a further and more serious slide occurred which resulted in the left bank abutment collapsing altogether and caused the recently completed 90 ft. approach span to fall into the river bed, throwing the bridge out of action for wheeled traffic. The condition of the bridge after the 1931 floods is shown in the sketch on page 16 and photograph No. 1.

Several schemes for repair or reconstruction were considered by the Punjab Government who finally accepted the author's design on the score of economy, rapidity of execution, its suitability for the peculiar condition at the right abutment and also because it enabled the reconstruction to proceed without interruption to pedestrian and cooly traffic. This scheme, which is described below, and illustrated on plate No.7 and the photograph, consisted of a special span cantilevered from the main river span to replace the wrecked 90 ft. approach span, and a light span suspended between a new Trestle abutment on the Punjab side and the nose of the cantilever. Details of the attachment of this suspended span to the nose of the cantilever are shown in plate No. 7. Its support on the trestle affords location in position, but not in direction. These provisions allow for any further movement likely to occur at the right bank and are designed to prevent any distortion in the cantilever span as in the event of forward movement, the suspended span would ride up on the cantilever nose. The roadway is composed of timber on the suspended and cantilever spans and for the first two bays of the main span. This, combined with the provision of new single rocker bearings on the main pier supporting the cantilever which centralise the reaction, keeps the compressive stresses on this pier under dead load to below the original figure.

The main problem in connection with the scheme was that of obtaining the necessary stiffness. Cantilever extensions of adequate strength could have been built out from the existing girders of the river span which would not be overstressed under the greatest live load using the road, but the deflection and oscillation would have been excessive and it was decided to limit the maximum static deflection at the cantilever nose to $\frac{3}{4}$" under the heaviest vehicle likely to be required to run on the bridge. The extra stiffness in the main river span was secured by the addition of a third girder and by building a 70 ft. cantilever from each, extending towards the Punjab side.

When the details of the structure had been worked out, the deflection of the cantilever nose due to one-ton axle load located at that point was calculated to be ·08 inches. This compares well with the actual deflection under the test load of 8 tons which was ·625 inches.

Execution of the work.

The first operation was to erect a special light service span extending from the main span to the bank on the Punjab side. This span which is shown on photograph No. 3 was supported by one main pier and by a steel service trestle, founded in the river bed near to the site of the old abutment, and was cantilevered forward to extend to the left bank over the site of the old abutment and the portion of the river bed which had spalled away on the collapse of the abutment during the landslide in July, 1931. This service span fulfilled several functions. Suitable lifting tackle was suspended from it to raise the wrecked 90′ approach span from the river bed in convenient units and it also acted as an erection gantry for the new 70′ cantilever span which has replaced the wrecked 90′ span and facilitated the transport of strengthening materials for the main river span and throughout the reconstruction it also functioned as a highway for pedestrians and coolies. The foot traffic was not interfered with at any stage of the reconstruction.

A 130′ girder surplus to railway requirements and similar to those of the main river span was available with the North-Western Railway. One complete cantilever girder was fabricated and assembled as an extension to the surplus 130′ girder in the North-Western Railway Bridge Workshop at Jhelum. This was then transported in convenient lengths to the bridge site. During the erection of the temporary service span, a convenient level space about 30 ft. by 100 ft. was cleared near the right abutment and was used as a fabricating yard for the second and third cantilever girders which were quickly made up entirely out of serviceable sections from the damaged 90 ft. approach span. The erection of the third girder for the river span was carried out on this span itself after the existing and temporary service spans had been slewed 4 ft. transversely in the downstream direction. Owing to the very limited space on the pier tops, special erection steelwork was used for slewing and lowering the new girder to its final position on the piers. Details of this service work including the jacking arrangements for lowering. are illustrated on plate No. 7 and on photograph No. 2.

On completion of this operation, the cantilever extension was erected from the temporary span and the original 130′ girders were slewed into their correct positions. At this stage fabrication work on the second and third cantilevers was well advanced. These were also then erected from the temporary span and braced together. The timbering of the roadway deck on the cantilever span was then taken in hand and the timber trestle on the Punjab side was completed to take the approach girder work including the 15′ suspended span which were all fabricated at site.

A general view of the completed bridge at the time of testing is shown on photograph No. 4.

Protection work at the Punjab side abutment.

In order to check, as far as possible, any further movement of the protection work, certain measures to reinforce the stone revetment were undertaken by the Punjab Public Works Department last winter during the period of low river level.

Near the foot of the river pier on the Punjab side, the rock in the river bed was dressed off to form a seating for a substantial masonry footing which forms the base for a dry stone apron covered with galvanised No. 4 I. S. W. G. steel wire trangars extending up to the old abutment and from that level to the new trestle abutment. Details of this masonry work are shown on plate No. I. The writer had originally recommended a rail pile footing composed of about fifty piles spread over an area of 60′×20′ for this apron in the river bed. These piles consist of a pair of double-headed 75 lbs. rails 24′ in length rivetted together and with the ends sharpened at the nose. They are capable of being satisfactorily driven down in to a boulder bed, and were used by the author with success in protecting the pier foundations of the Michni bridge across the Kabul River on the Shabkader Road. As, however, for this scheme it would have been necessary to transport all the piles and a steam-boiler and piling hammer to the site, the writer has very little doubt that the masonry apron designed and constructed by the Public Works Department will fulfil all requirements.

Transport of Plant and materials to site.

The new steel work and all tools and plant were despatched from the Jhelum Bridge Depot to Rawalpindi by rail and from there by road to Kohala by the Military Department using 6 tons 6-wheel motor lorries. The longest length dealt with in a single unit was 25 feet. The rapidity with which the transport work was undertaken was an important factor in the early completion of the work.

The job was completed in three and a half months from the time the proposal was mooted and this included the time spent in preparing the detailed design.

The bridge was re-opened to traffic on the 1st August, 1932.

PLATE № I

FIG. 2

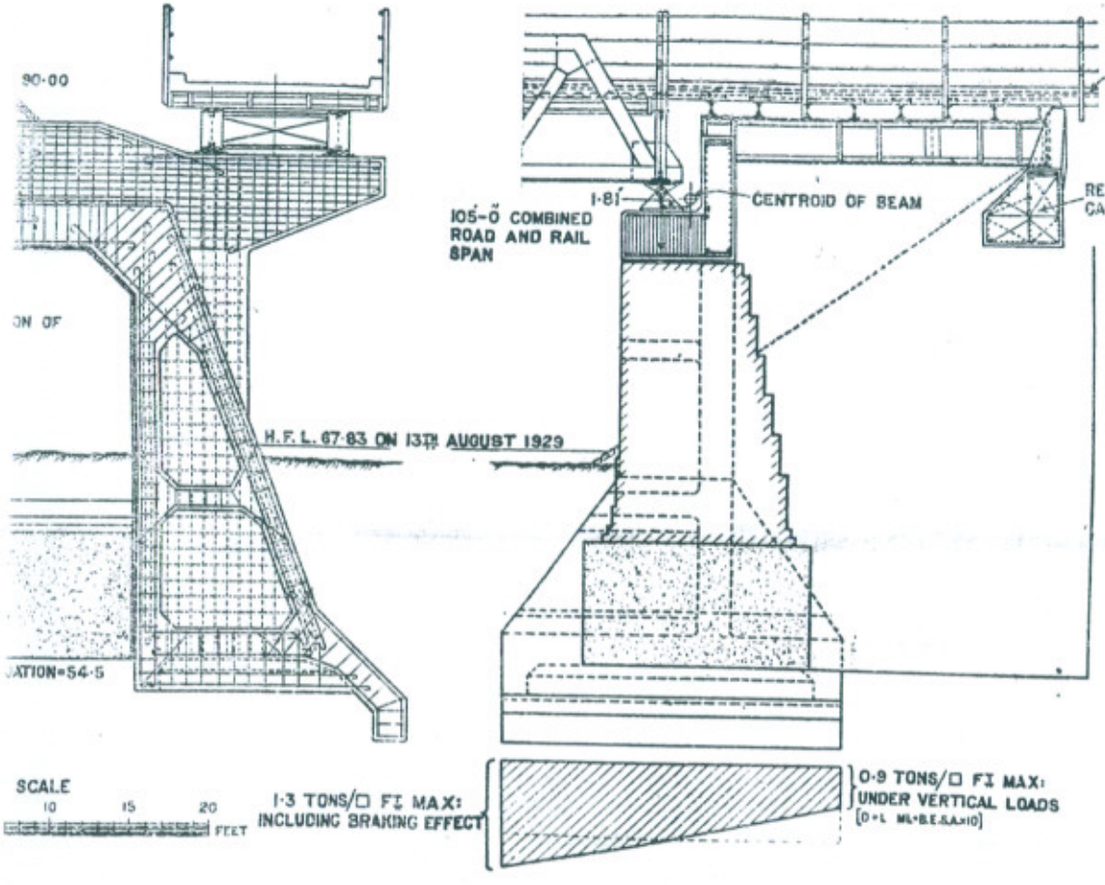
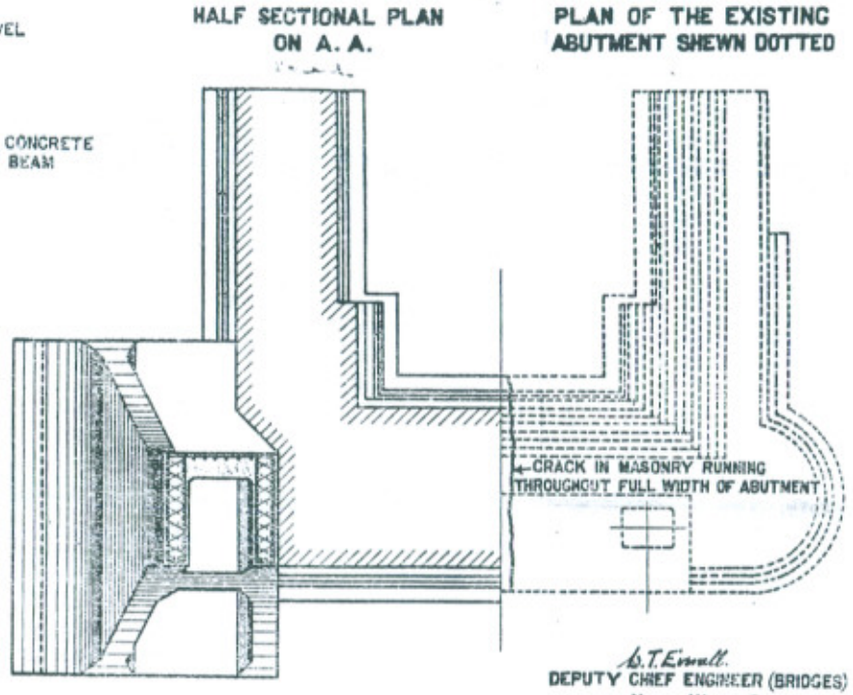


FIG. I



A.T. Emmett.
DEPUTY CHIEF ENGINEER (BRIDGES)
N. W. R.

INDUS BRIDGE AT KOTRI
REINFORCEMENT OF ABUTMENT ON THE APPROACH SPAN
CARRYING COMBINED RAIL AND ROAD TRAFFIC

FIG. 3

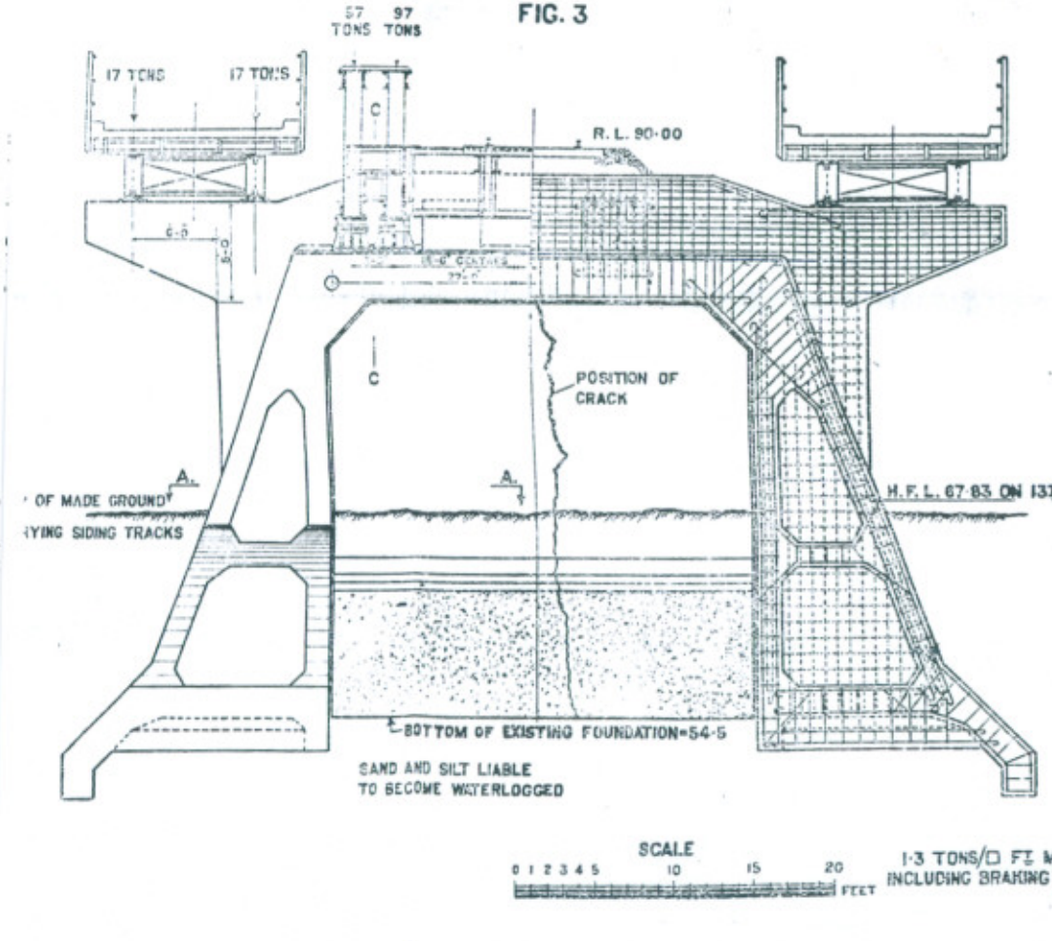
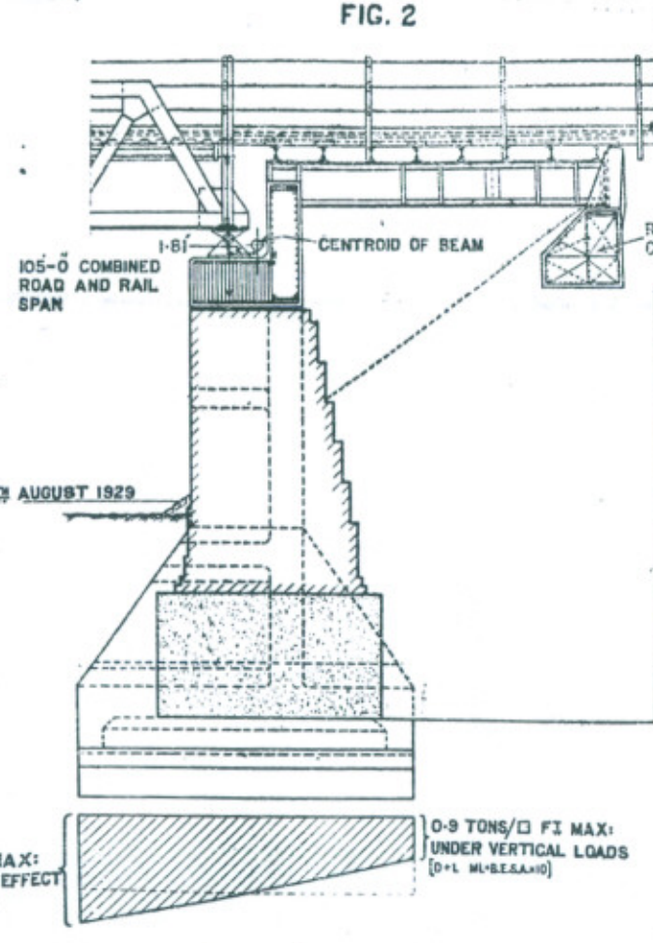
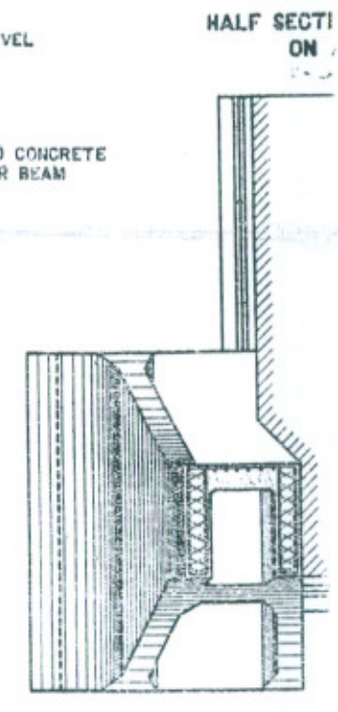


FIG. 2





INDUS
REINFORCEMENT OF A
CARRYING COMBIN

CHENAB BRIDGE AT CHINIOT

SPECIAL REINFORCEMENT IN WELL & PIER CAP

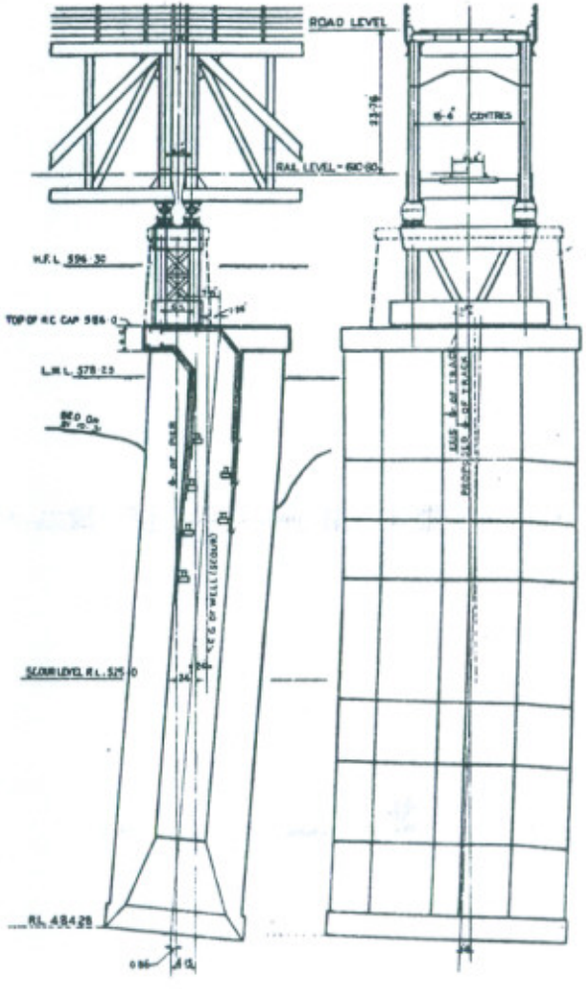
PLATE Nº 2

SCALES

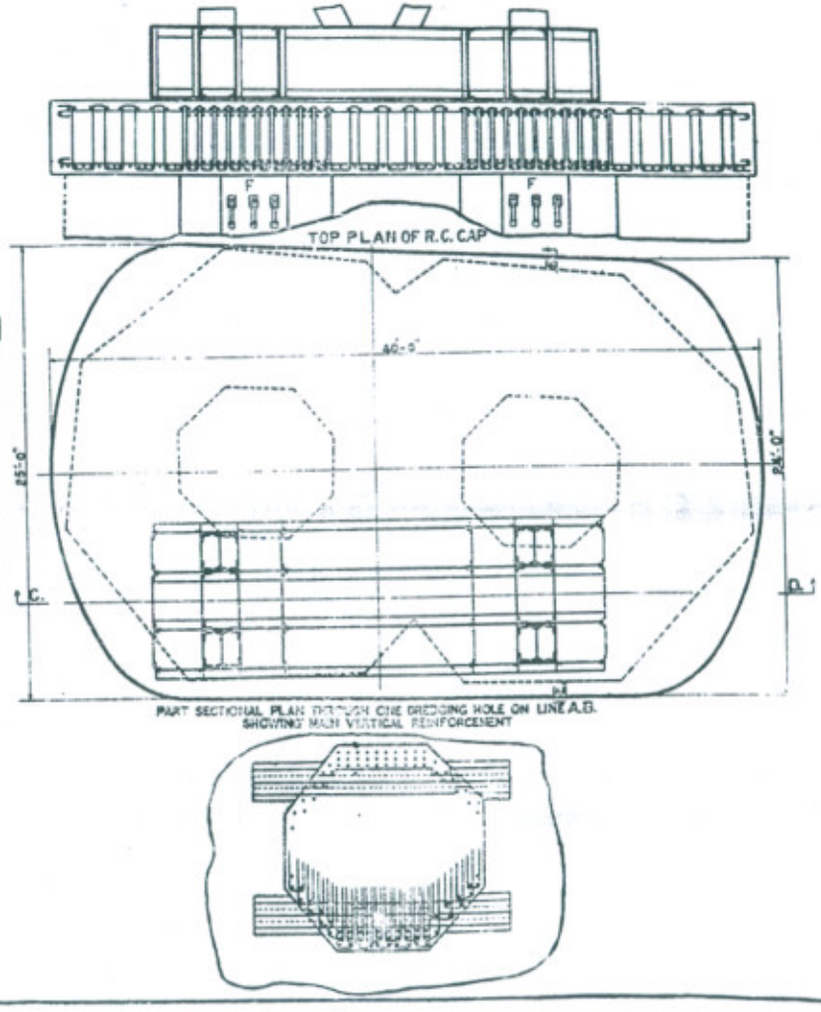
FEET 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18

FEET 0 2 4 6 8 10 12 14

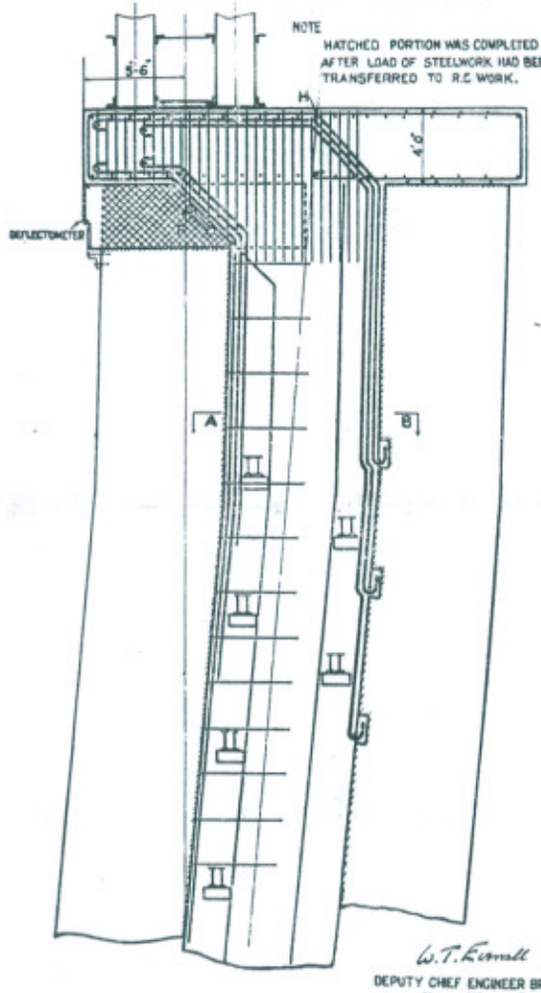
FEET 0 2 4 6 8 10



SECTION OF CAP ON C.D.



PART SECTION OF WELL ON E.F.

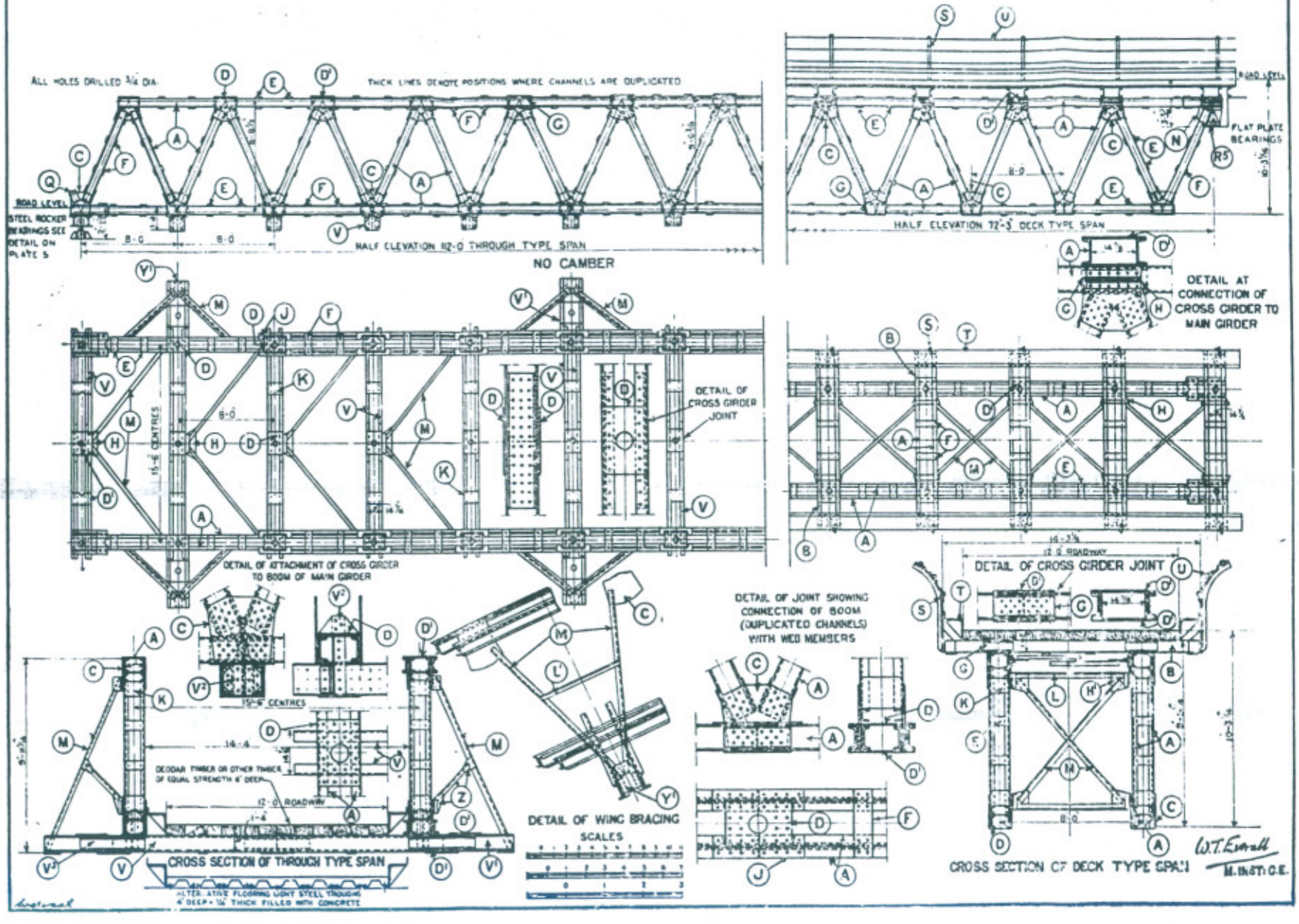


W. T. Linnell

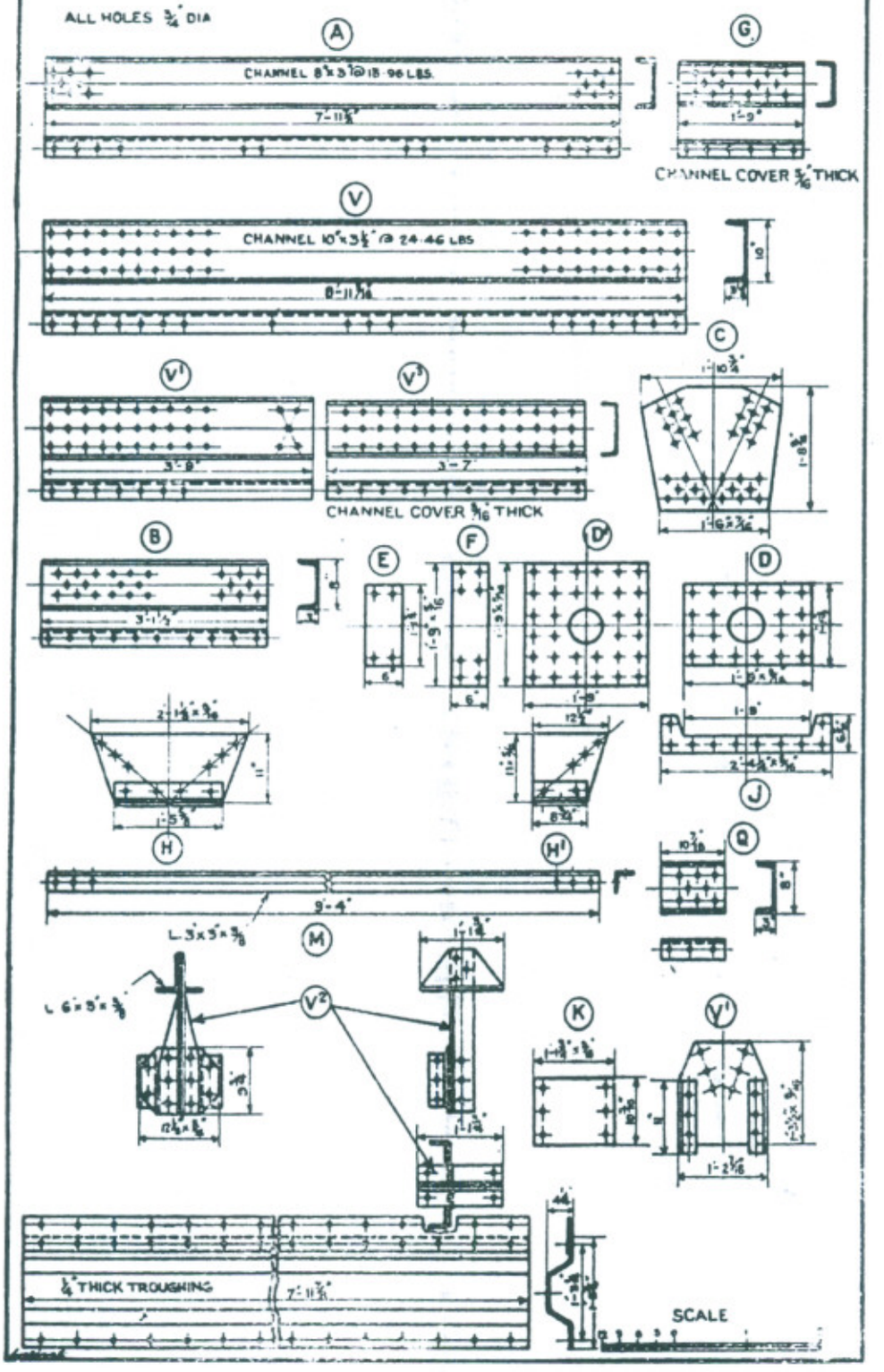
DEPUTY CHIEF ENGINEER BRIDGES
N. W. R.

PATENT APPLIED FOR IN INDIA
AND UNITED KINGDOM

PORTABLE ROAD BRIDGE SPANS
ARRANGEMENT OF DECK AND THROUGH TYPES
BUILT OF STANDARDIZED INTERCHANGEABLE UNITS

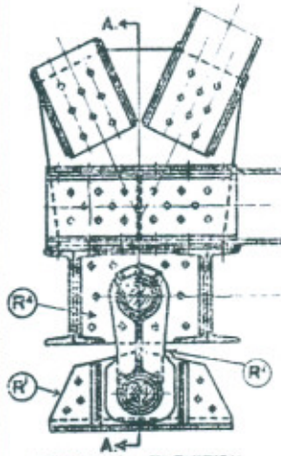


DETAILS OF STANDARD UNITS FOR THROUCH AND DECK TYPE GIRDERS

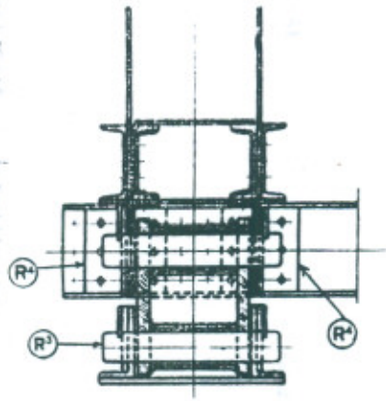


DETAILS OF STEEL ROCKER BEARINGS FOR THROUCH AND DECK TYPE GIRDERS

EXPANSION BEARINGS FOR THROUCH TYPE SPAN

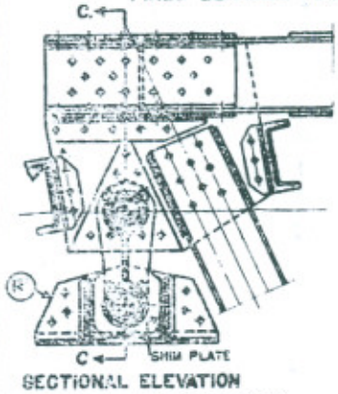


SECTIONAL ELEVATION

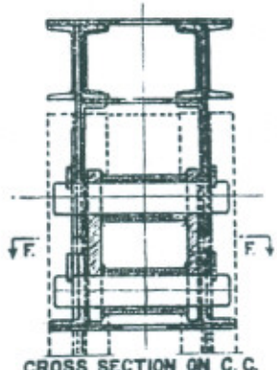


CROSS SECTION ON A A.

FIXED BEARINGS FOR DECK TYPE SPAN

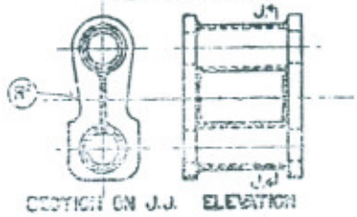


SECTIONAL ELEVATION

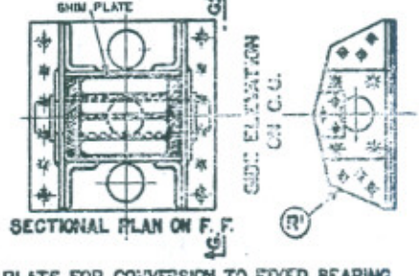


CROSS SECTION ON C.C.

DETAIL OF LINK



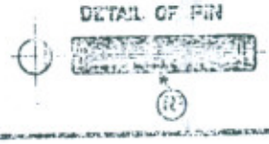
SECTION ON J.J. ELEVATION



SECTIONAL PLAN ON F.F.

SHIM PLATE FOR CONVERSION TO FIXED BEARING

DETAIL OF PIN





PORTABLE ROAD BRIDGE SPANS (56 TO 120 FEET GIRDERS)

PARTICULARS OF WEIGHT, NUMBER AND MARKING OF STANDARDIZED UNITS FOR TYPICAL DECK & THROUGH SPANS

PERMISSIBLE LOADING

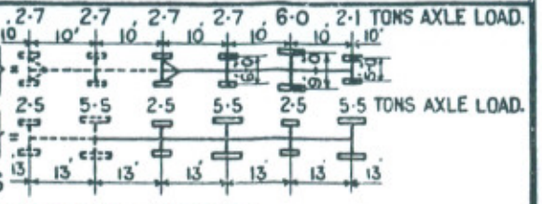
(1) BRITISH STANDARD ASSOCIATION X. 6 UNITS (ROADWAY FULLY COVERED)

OR

(2) TABLE "F" ON PAGE 199 OF M. E. VOL III (ROADWAY FULLY COVERED)

OR

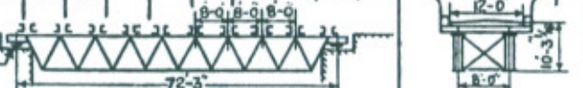
(3) SINGLE UNIT VEHICLE OR TANK OF THE FOLLOWING LIMITING WEIGHTS AND DIMENSIONS. 7 TONS 7 TONS AXLE LOAD. 8-6 CENTRES



IMPACT ALLOWANCE ADDITION { 25% FOR MAIN GIRDERS; 50% FOR FLOORING UNITS

SPAN CENTRES OF BEARINGS	MARKING OF UNIT A	B	C	D	D'	E	F	G	H	H'	J	K	L	L'	M	N	BEARINGS Q	R1	R2	R3	R4	R5	R5a	S	T	U	V	V'	V2	V3	Y	Y'	Z	STEEL DECK TROUGH UNIT INCLUDING WASHERS & BOLTS
	WEIGHT OF EACH UNIT LBS:																																	198 LBS PER UNIT
	127·5	66·7	54·7	25·5	39·0	7·3	11·1	22·5	50·4	24·6	8·8	12·4	44·2	35·8	66·9	119·9	13·9	226·4	125·5	54·3	81·3	93·3	79·0	58·1	210·0	19·6	218·7	91·7	78·0	71·3	7·5	37·7	43·9	
72'-3"	NUMBER OF UNITS IN 72'-3" SPAN OF DECK TYPE																																	NUMBERS
	160	48	68	76	44	216	56	116	20	16	NIL	144	8	NIL	26	8	32	NIL	NIL	NIL	NIL	4	NIL	20	19	57	NIL	NIL	NIL	NIL	NIL	NIL	NIL	72
	No 6768 = 3/4" DIA: BOLTS & NUTS × 2" LONG } = 2·83 TONS INCLUDED IN TOTAL WEIGHT OF SPAN.																																	
	" 576 = 3/4" DIA: BOLTS & NUTS × 2 3/4" LONG																																	
	" 3969 = 3/4" TAPPER WASHERS SQUARE																																	
	" 2772 = 3/4" PLAIN WASHERS ROUND																																	
	" 576 = 3/4" DRIFTS × 3 1/2" LONG																																	

FLAT PLATE BEARINGS



TOTAL WEIGHT OF SPAN ---------- = 23·00 TONS.
ADDITIONAL WEIGHT OF DECK TROUGHING = 6·3 TONS.

112'-0"	NUMBER OF UNITS IN 112'-0" SPAN OF THROUGH TYPE																																	
	312	8	116	110	78	192	184	200	6	12	84	284	NIL	8	28	NIL	40	4	4	8	8	NIL	NIL	NIL	29	NIL	68	16	26	64	16	8	16	112
	No 11000 = 3/4" DIA: BOLTS & NUTS × 2" LONG } = 4·74 TONS INCLUDED IN TOTAL WEIGHT OF SPAN.																																	
	" 1556 = 3/4" DIA: BOLTS & NUTS × 2 3/4" LONG																																	
	" 7000 = 3/4" TAPER WASHERS SQUARE																																	
	" 2156 = 3/4" PLAIN WASHERS ROUND																																	
	" 868 = 3/4" DRIFTS × 3 1/2" LONG																																	

ROCKER BEARINGS



TOTAL WEIGHT OF SPAN ---------- = 47·60 TONS.
ADDITIONAL WEIGHT OF DECK TROUGHING = 9·8 TONS.

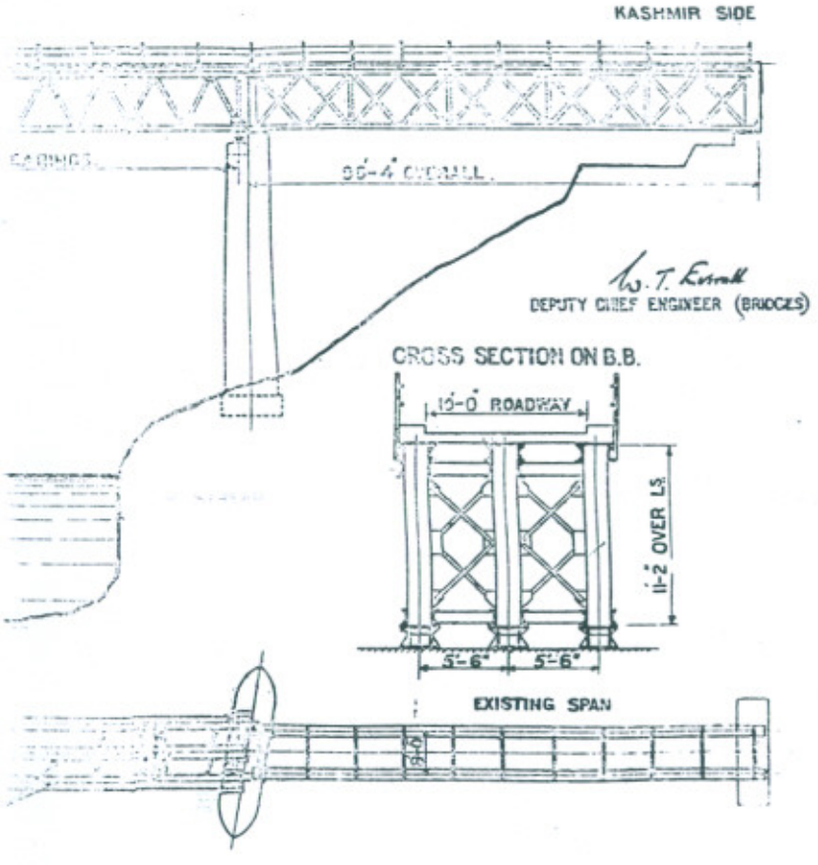
20'-3"	NUMBER OF UNITS IN 120'-3" SPAN OF DECK TYPE																																	
	372	72	116	106	86	208	264	252	32	24	76	240	12	NIL	42	8	40	4	4	8	NIL	NIL	NIL	32	31	93	NIL	NIL	NIL	NIL	NIL	NIL	NIL	120
	No 11925 = 3/4" DIA: BOLTS & NUTS × 2" LONG } = 4·97 TONS INCLUDED IN TOTAL WEIGHT OF SPAN.																																	
	" 1350 = 3/4" DIA: BOLTS & NUTS × 2 3/4" LONG																																	
	" 4845 = 3/4" TAPER WASHERS SQUARE																																	
	" 4620 = 3/4" PLAIN WASHERS ROUND																																	
	" 930 = 3/4" DRIFTS × 3 1/2" LONG																																	

ROCKER BEARINGS



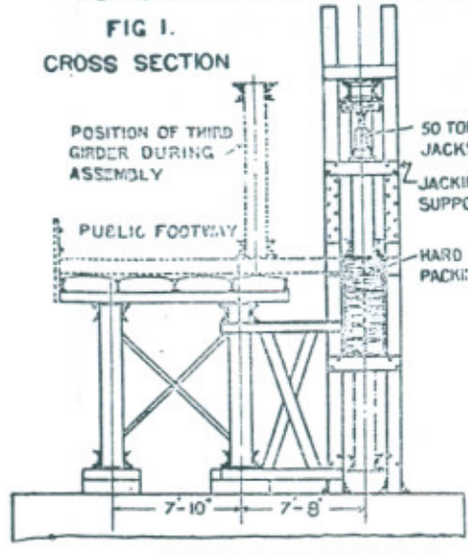
TOTAL WEIGHT OF SPAN ---------- = 45·46 TONS.
ADDITIONAL WEIGHT OF DECK TROUGHING = 10·5 TONS.

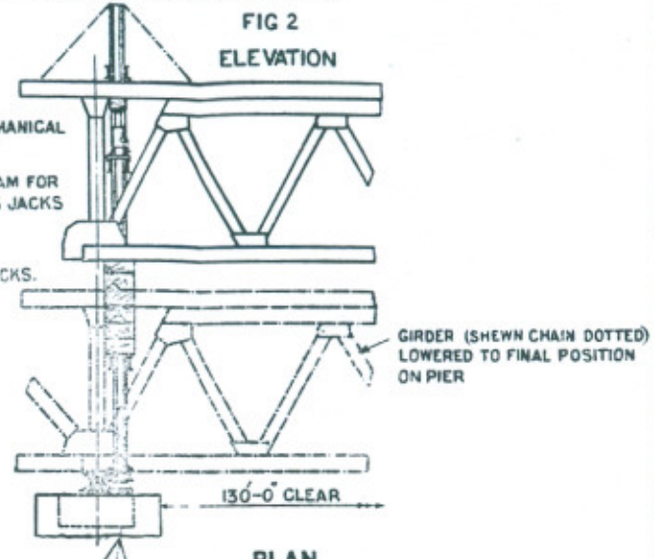
BRIDGE
AD

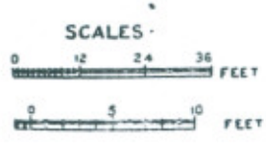


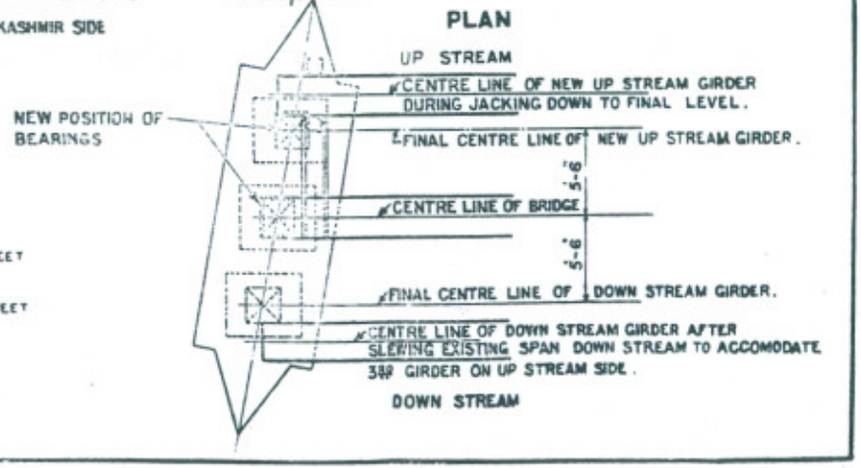
TEMPORARY SERVICE TRESTLES ERECTED ON EACH MAIN PIER FOR SLEWING & LOWERING THIRD MAIN GIRDER FOR 130 Ft. SPAN

PLATE № 7

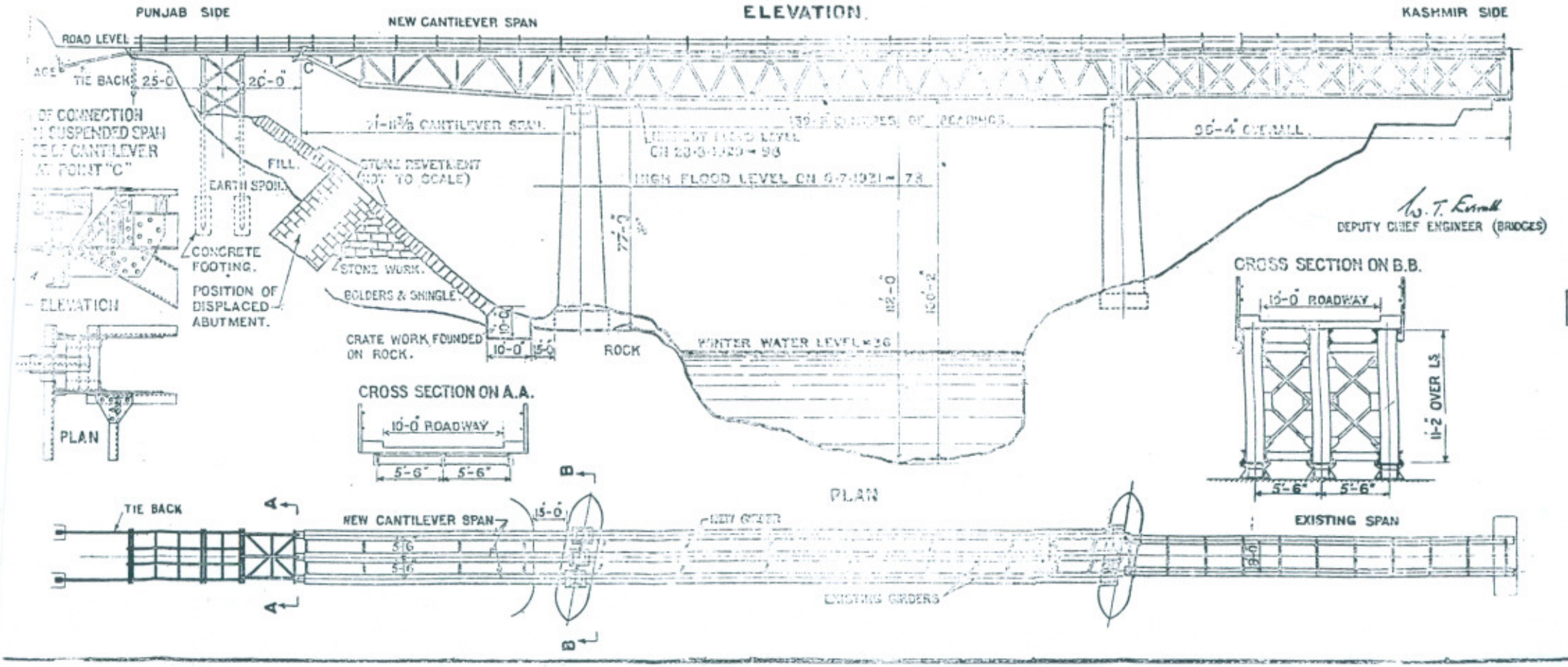








RECONSTRUCTION OF THE KOHALA BRIDGE ON THE RAWALPINDI KASHMIR ROAD



KOTRI BRIDGE PHOTO No. 1.

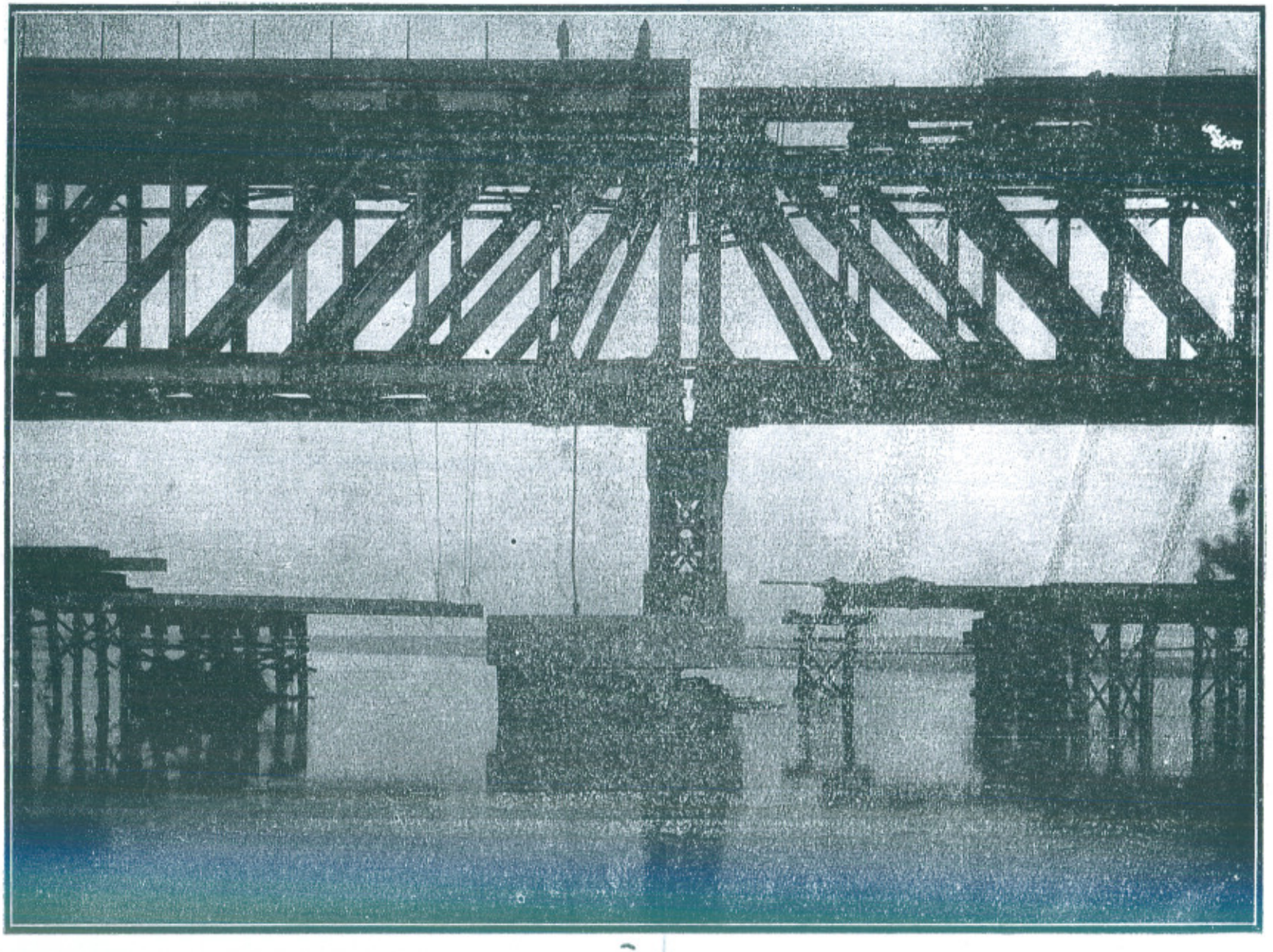


PORTIQN OF REINFORCEMENT IN POSITION.

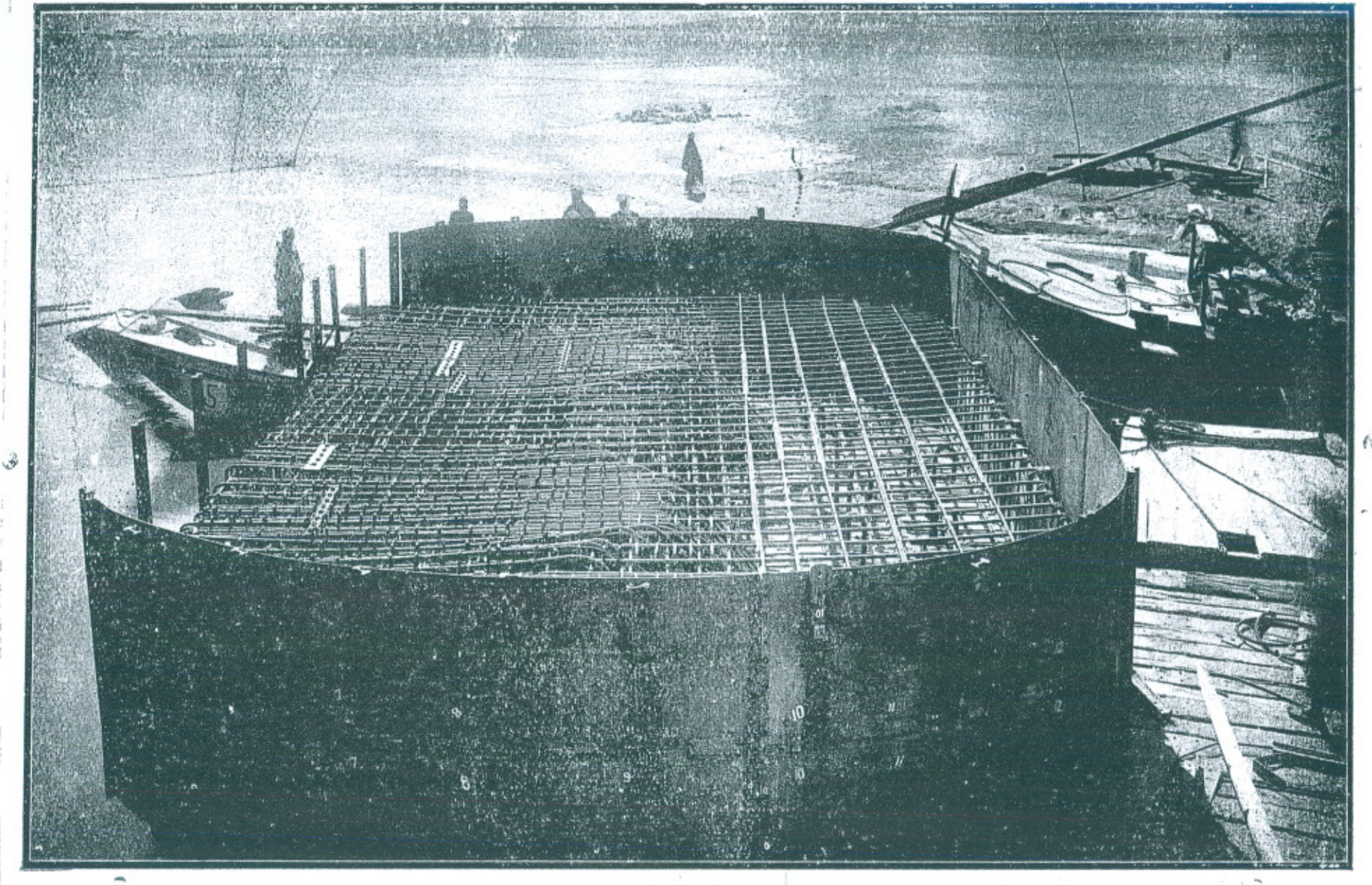
KOTRI BRIDGE PHOTO No. 2.



CHINIOT BRIDGE PHOTO No. 1.

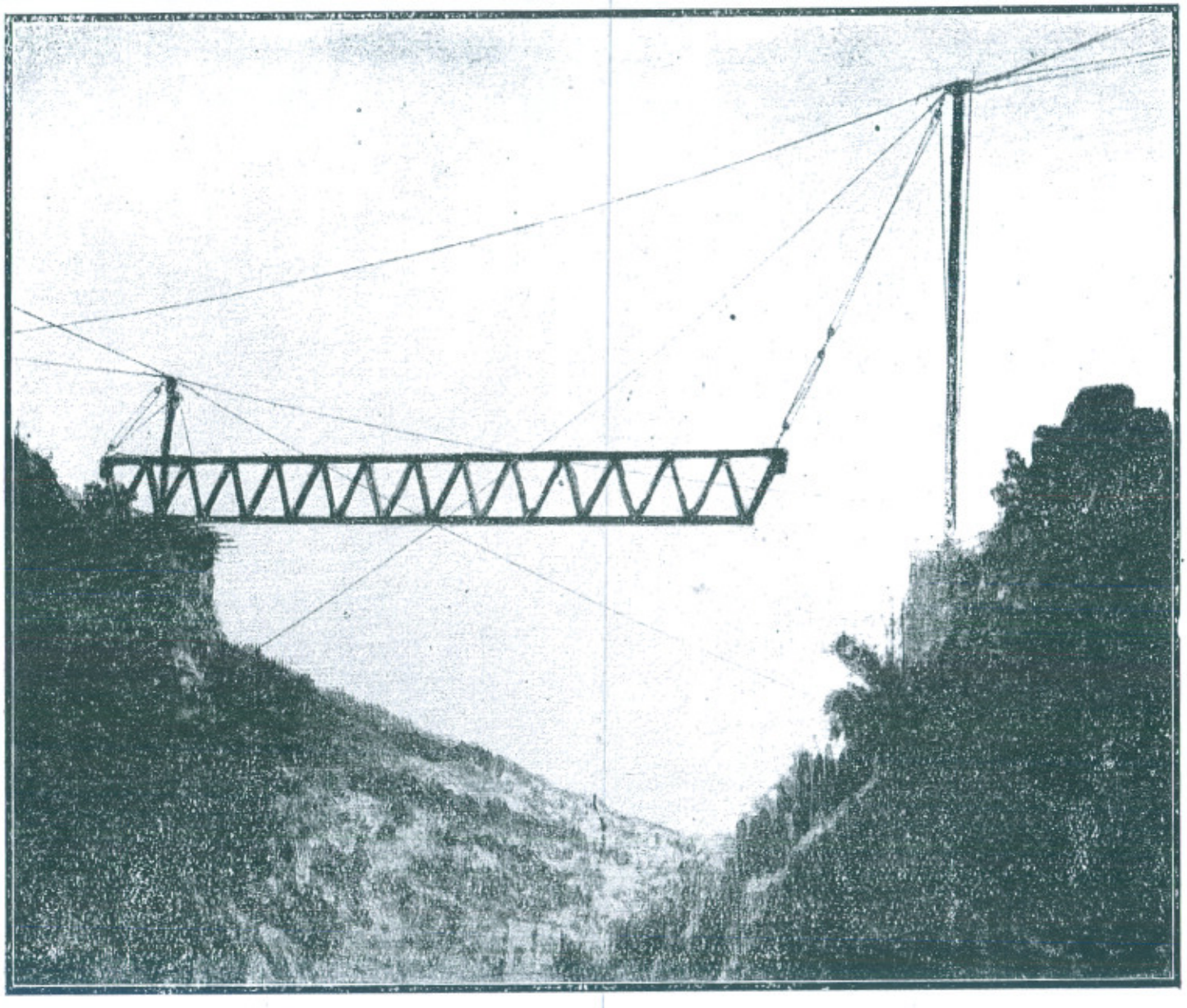


ELEVATION AT WELL No. 4 WITH STAGING REMOVED.

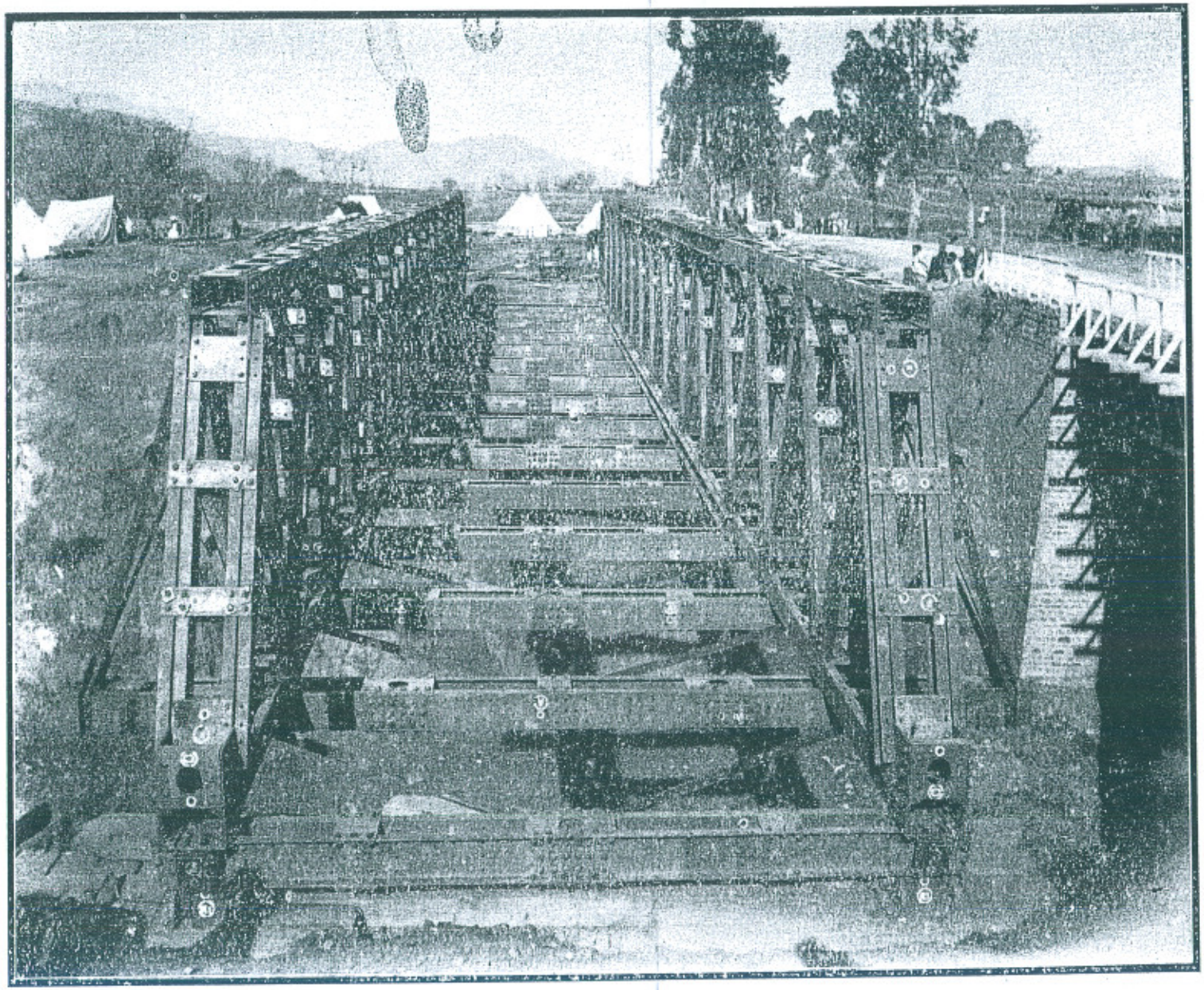


VIEW SHOWING REINFORCEMENT COMPLETELY ASSEMBLED FOR CONCRETING.

PHOTO No. 1.

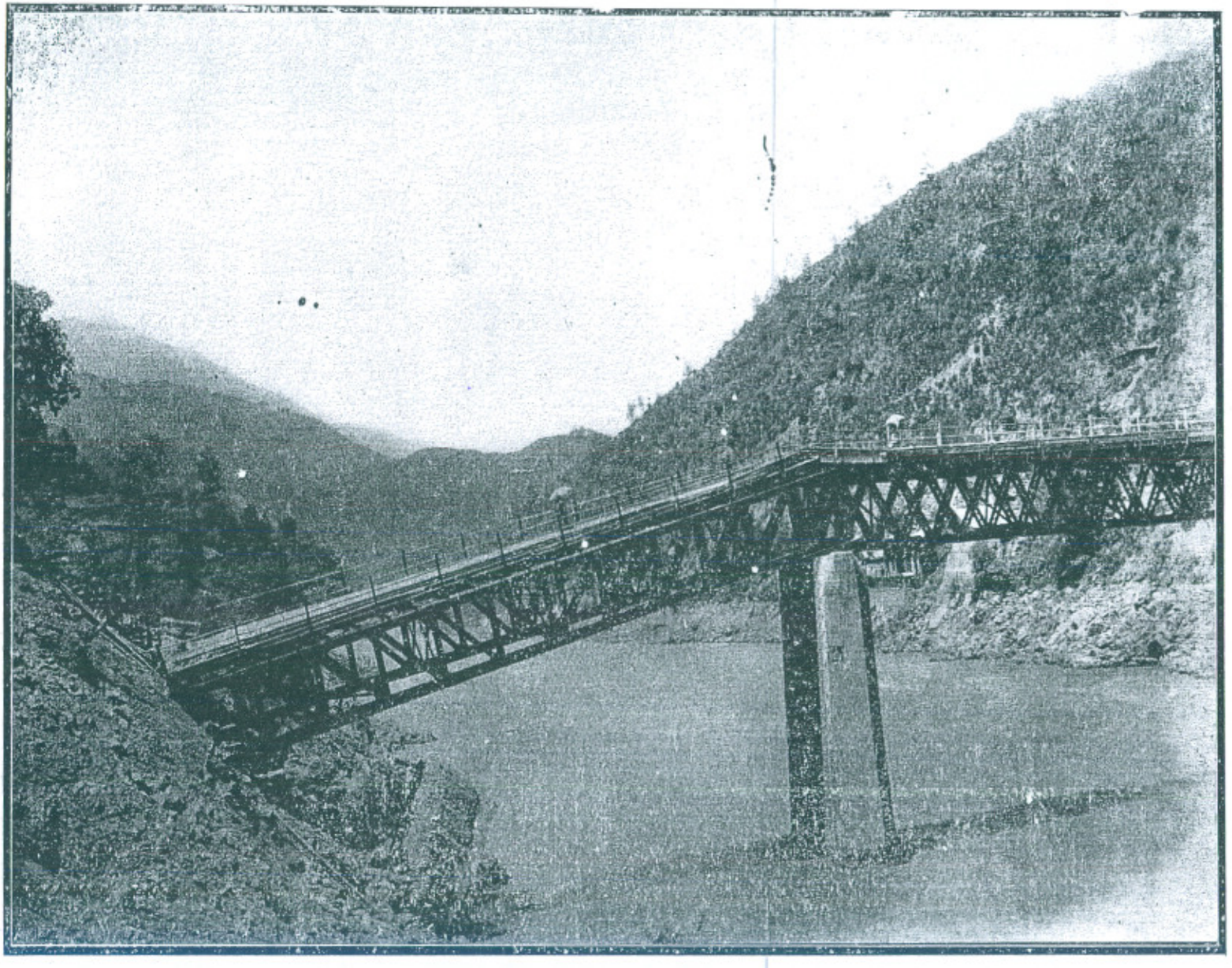


ERECTION OF MAIN GIRDER OF 112′ DECK SPAN OF THE CHUTIATAN BRIDGE ON THE CHITRAL ROAD.



112′ THROUGH TYPE SPAN ERECTED ON THE MANSAHRA DIVISION HAZARA N. W. F. P.

KOHALA BRIDGE PHOTO No. 1.



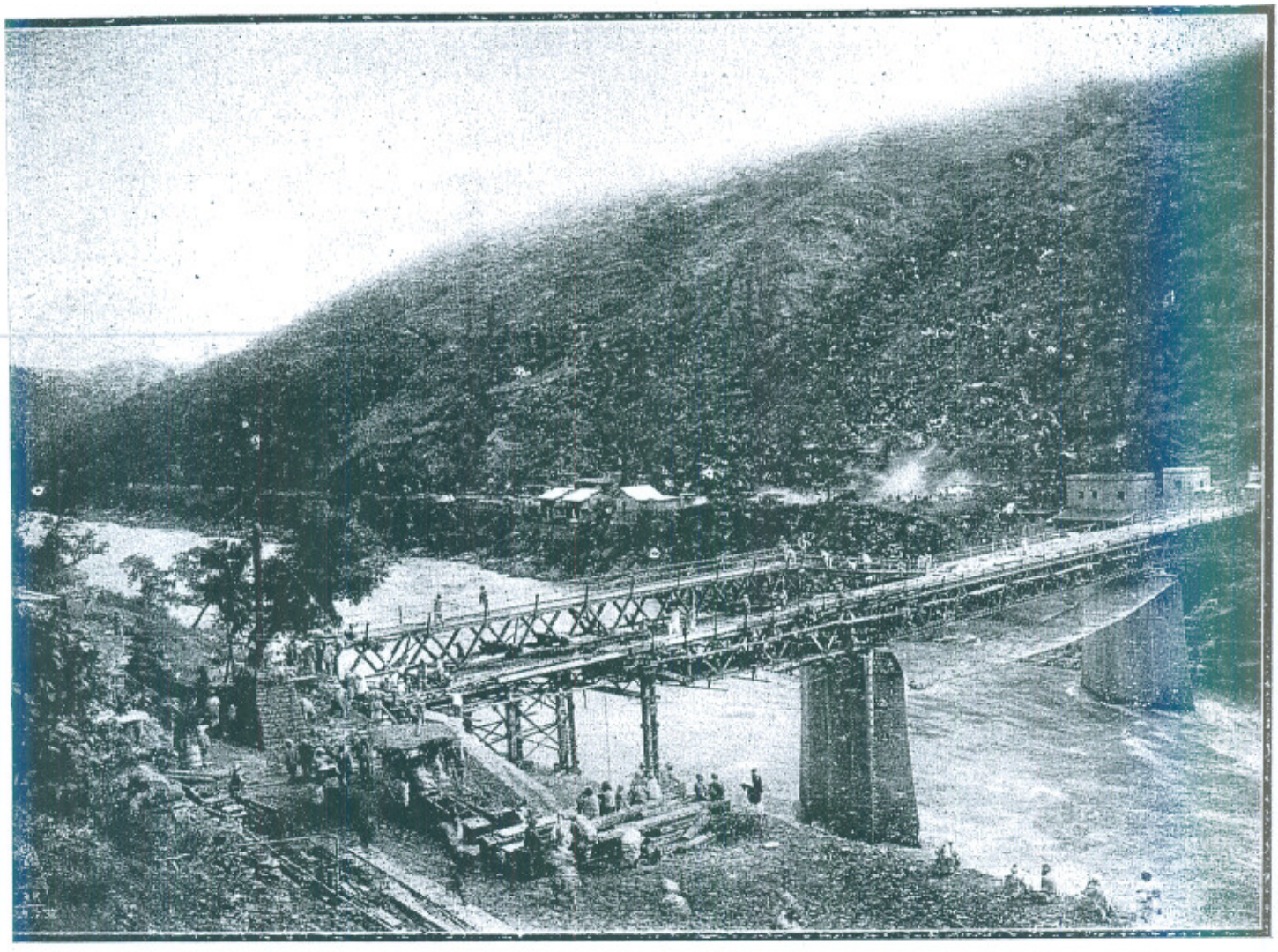
POSITION OF WRECKED ABUTMENT AND APPROACH SPAN AFTER FLOODS OF 6th JULY 1931.

KOHALA BRIDGE PHOTO No. 2.



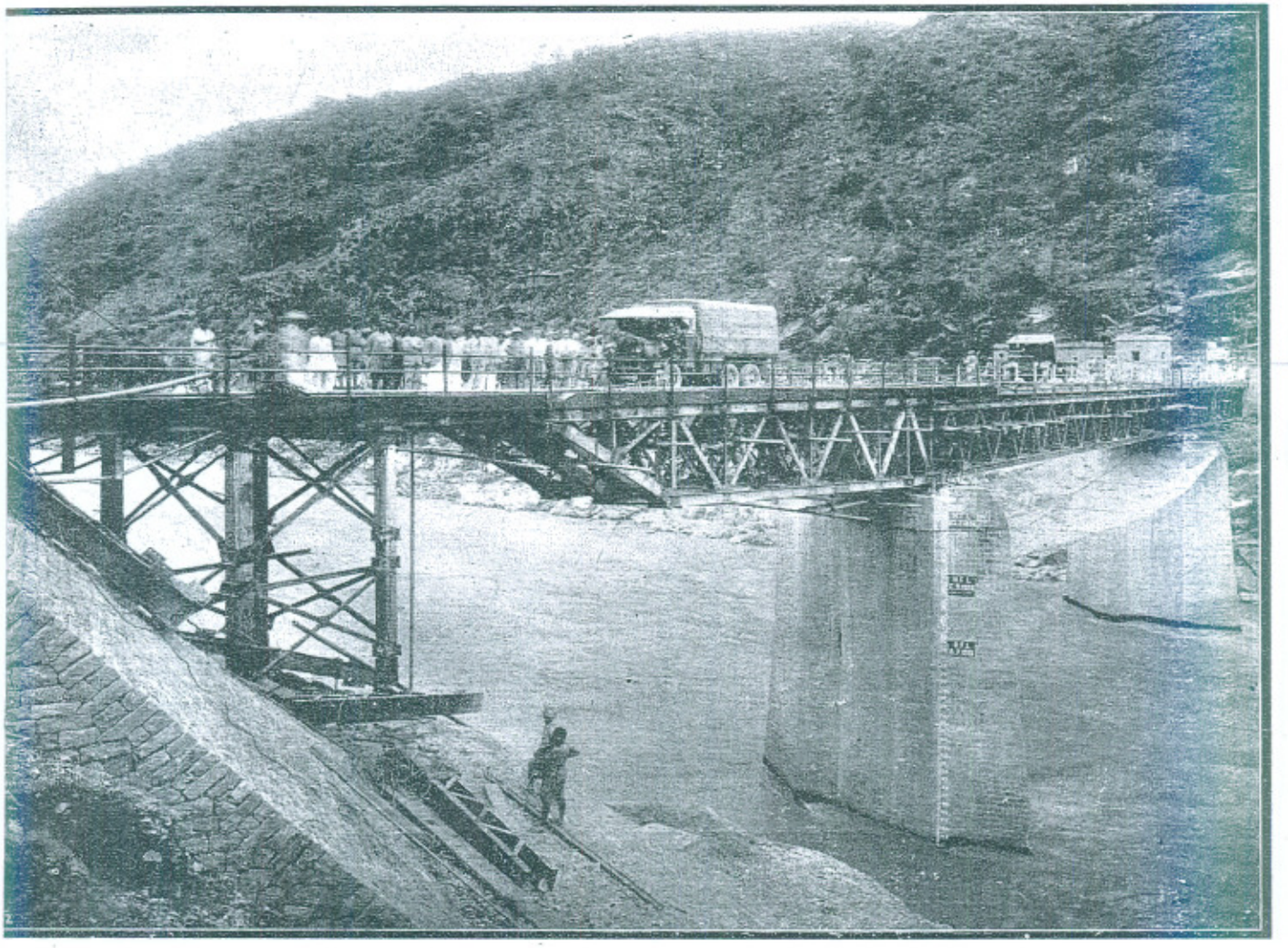
VIEW SHOWING SERVICE WORK AND JACKING ARRANGEMENTS FOR ERECTION OF GIRDER OF RIVER SPAN.

KOHALA BRIDGE PHOTO No. 3



SERVICE SPAN AND TRESTLE IN POSITION AND ERECTION OF CANTILEVERS WELL ADVANCED.

KOHALA BRIDGE PHOTO No. 4



BRIDGE COMPLETED SHOWING 7 TON MOTOR LORRY USED FOR TESTING.

DISCUSSION.

Mr. Everall in presenting this Paper said that he thought the members might be interested in a description of several features of bridge construction which were somewhat unusual and not met with in every-day practice.

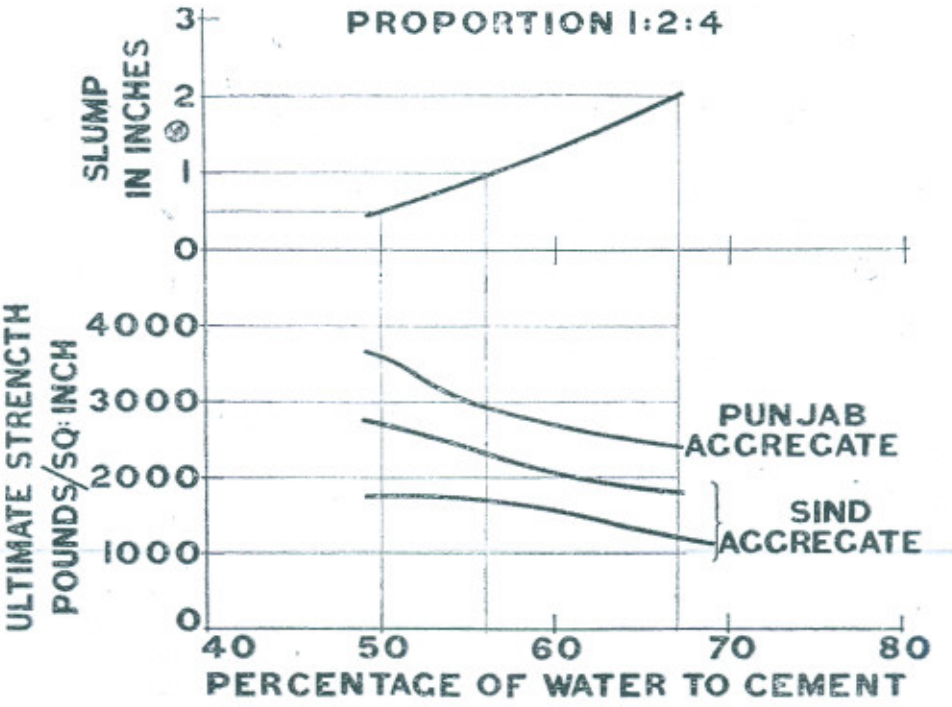
In placing these features before the members he wished to say that there were many ways of tackling each problem and although the ones adopted fulfilled their objective, other methods would possibly have been equally satisfactory. The description of the various works had of necessity to be condensed and he had, therefore not been able to include all the information which he would like to have placed before the members. For instance, in describing feature No. 1.—The reinforcement of a cracked abutment on the approach span of the Kotri Bridge, the reinforcing rods were electrically butt welded together to form lengths of from 40 to 60 feet, these rods were received at Kotri in short lengths of 16 feet. The electric welding of the reinforcing rods was a departure from the usual practice in concrete work in India; it had advantages in avoiding the use of lap joints and provided considerably more space for placing the aggregate between closely spaced rods. The welding was done with mild steel Quasi Arc Electrodes and the specimens which were tested broke some distance away from the welded joint itself. The rods were also bent over cold through 180° and gave satisfactory results.

Another matter of interest was in connection with the sand and stone aggregate used on this work. It was found after examination and testing of many samples of sand and stone available in Sind that these materials were not suitable for making high grade reinforced concrete such as was required for the work on this abutment or for structures subjected to heavy dynamic loads. It was found necessary to import from the Punjab, the sand and stone aggregate used (the sand from Lawrencepur near the Haro River and the broken stone from Wah). Compression tests on 4" dia. cylinders gave results 30% higher, when using Punjab sand and stone aggregate than those obtained from Sind using sand from Bolari and shingle from Landhi near Karachi. The results obtained from typical tests are shown on the diagram below :—

For the reinforced concrete a slump of 2" was the lowest that gave a reasonable workable mixture, and with this slump the Punjab aggregate gave 2400 lbs. per square inch as against 1800 lbs. per square inch from the best materials obtained in Sind.

The Bridge Branch had discarded the use of Sind sand and stone for all their important bridge works, such as seating blocks, reinforced concrete bridge slabs and reinforced concrete abutments and piers and now imported sand and stone from the Punjab and Baluchistan.

DIAGRAMS SHOWING RELATION BETWEEN WATER-CEMENT RATIO SLUMP AND ULTIMATE STRENGTH IN COMPRESSION AT 28 DAYS OF CONCRETE MADE FROM SIND AND PUNJAB AGGREGATE AND CEMENT FROM WAH



Regarding Features No. 2.—Reinforcing of a tilted well of the Chenab Bridge at Chiniot.

The Author was unable to give detailed information regarding the sinking of this well as his work on the bridge had been limited to the superstructure. The reinforcing of the well described in the paper had been undertaken at a later stage when practically all the other wells had been sunk to their final positions. The Bridge staff were fortunate in being able to carry out tests on the slab after its completion and to record measurements of the stresses with recording instruments. This was made possible by having the staging in position beneath the main girders, and from this staging mechanical jacks were placed for applying the 500-tons load to the slab. Under these favourable conditions it had been possible to apply the load and take it off several times and this enables a check to be made on the measurements over a fair number of observations.

Mr. Everall remarked that he wished to mention the valuable assistance which he had received from Mr. Gelson, Assistant Bridge Engineer, in connection with these tests and also in the design of the concrete structures.

Feature No. 3.—In connection with the portable type Road bridge, these structures are now in use on the Frontier Roads and had been adopted by the Army as one of their Standard patterns. Sapper and

Miner Companies were being trained out in the assembly and erection of these bridges. He had described this bridge because he thought that the members may be interested to know something of its developments from a structure originally designed for an unusual purpose and its subsequent development into a standard type of bridge for Military use.

The last item dealt with was the Kohala bridge. Mr. Everall stated that his note did not do justice to the work done by the P. W. D. staff and the Military in connection with this bridge. The Paper had stressed the erection of the girders but very little mention was made of the work done by the P.W.D. staff. The latter placed the heavy stone revetment in position below the trestle on the right bank. They also carried out all the construction of the timber flooring on the bridge and arranged for the road transport of the materials from Rawalpindi to the bridge site, in conjunction with the Heavy Mechanical Transport Coy. of the Military Department, and in every possible way gave the writer full assistance and co-operation in carrying out his part of the work.

Mr. Everall in conclusion mentioned how much he appreciated the work done in this connection by Mr. Trevor Jones who was the Executive Engineer and later the Superintending Engineer of the Circle, and Mr. D. Macfarlane as Superintending Engineer, and later as the Chief Engineer of the Roads and Buildings Branch of the P. W. D. also the Heavy Mechanical Transport Company, I.A.S.C.; under the direction of Captain Marchant, the Transport Officer of the Rawalpindi-Murree Area.

Lieut.-Col. W. Macrae, R.E., in discussing Mr. Everall's paper, remarked that he happened to be able to give some information supplementing that in Mr. Everall's paper, leaving criticism to other members some of whom he hoped would give helpful criticism on those unusual structures.

Item No. 1.—The Kotri Bridge was built and opened in 1900, the piers and the left abutment being carried down to rock, only the abutment of the short shore span being on shallow open foundations; the span crossed the railway sidings serving the Indus River Bunder for river traffic at Kotri. As an open line Deputy Chief Engineer he had noted the cracked abutment with yearly tell-tales dating back nearly 20 years, all broken; as there was a speed restriction across this bridge and a scheme for girder re-construction and as the vertical crack was midway between the girders, it was decided to leave things alone until the girder reconstruction. He had become Chief Engineer, North-Western Railway, about the time girder reconstruction was approved by the Government of India, leaving open to the Bombay Government the decision whether roadways should be provided or not. Apart from the main girder design to carry roadways on cantilevers if required, roadway details had not been settled, a lump sum for approaches being provided in the estimate. On notice being given to the Bombay Government that tenders were

being asked for the main girders, their decision to pay proportionate share of heavier main girders and to pay for roadways was received, a last minute decision.

The question of the cracked abutment was then gone into and the speaker called for a design for holding the cracked abutment together with a reinforced concrete cap, and then for distributing increased foundation pressures, resulting in the design described by Mr. Everall, the work being carried out under his supervision by Mr. Heinig, Executive Engineer, and Mr. Blackwood, Assistant Executive Engineer.

In regard to the concrete tests, Col. Macrae said that the poor quality of most of the stone and sand in Sind was a well known fact on the North-Western Railway and it was that which led to the tests.

Good stone and sand was obtainable in some parts of Baluchistan, and there were a few places in Sind which were still worth testing, both stone quarries and sand pits, to save lead, but for the subsidiary work required to complete a 23 lakh job, time did not permit of the further tests.

Item No. 2.—As regards the tilted well at the Chenab Bridge near Chiniot, he regretted that it had to be admitted that the cause was not only flood damage during sinking, but poor supervision by the construction Executive Engineer and bad work by his foreman, neither of whom were now in Government service.

One of the two sections showed clearly in plate No. 2 that the Pier was kinked as well as tilted. Apparently, when the foreman found difficulty in straightening the well, he had the masonry built more or less vertically above the tilted section already sunk, and gave his Executive Engineer to understand that he had straightened the well, the latter accepting that without proper check. He further remarked that the Chief Engineer, Surveys and Constructions, called for a report as to how the straightening of the tilted well had been done, and was not satisfied with the Executive Engineer's report. The latter went on leave due to him on termination of his contract. His successor, Mr. Thomson, who temporarily took over for six weeks at the end of the working season before the floods, found that further sinking increased the tilt inspite of heavy counterbalancing, ; he investigated the state of the well carefully and reported that the well had been built with a kink. On temporarily going to the Construction Branch during the flood season and taking over as Chief Engineer he received notes to that effect. Mr. Benton, who took over from Mr. Thomson after well sinking was stopped for the flood season, referred for orders the question of further sinking, having verified the facts discovered by Mr. Thomson and obtained additional information by sending down divers as pumping had failed to reduce water level sufficiently.

By this time the well was down to the depth shown on the Paper, and he concurred with Mr. Benton that no further sinking was possible except at the cost of increased tilt.

He reported matters and that he considered the depth attained to be enough to ensure safety. The Technical Member, Railway Board, came and inspected the bridge and the records concerned, the result being the unusual design described in the paper.

Item No. 3.—As regards item No. 3, it had been very interesting to watch the development of the experimental type of road girder into a standardised type for various lengths of spans with tables of units as shown on plate 6.

Some years ago one of Col. Macrae's own batch in the Sappers, now Chief Engineer, Southern Command, came to Lahore to discuss with him certain railway frontier problems, and one of his demands as his guest was whether he could meet Mr. Everall to get information about this type of bridge. Mr. Everall was unfortunately out on the line, but though not connected with this work officially Mr. Everall had discussed this scheme with him at intervals and he was able to give some information while he was supplied later with full details.

Item No. 4.—As regards item No. 4, Mr. Everall brought to him as Chief Engineer, North-Western Railway, his ingenious design for the Kohala Bridge for approval before submission to the P. W. D. and he inspected the work when it was being tested before opening, on his way back from casual leave.

The only criticism he had to make of the work was a suggestion for a very small improvement to ensure that the small shore span would slide up and over the main cantilever if the abutments slipped again, a matter that would largely depend on very careful maintenance of catchment drains.

In the work, the point chiefly caught the eye was the very narrow working space on the nose of the piers for supporting the extra girder when first lowered on to them.

Col. Macrae took an opportunity to mention that His Excellency the Governor of the Punjab sanctioned an honorarium for Mr. Everall for the work in question.

Mr. D. Macfarlane explained briefly the difficulties with which the B. and R. Branch were faced in connection with the Kohala Bridge and the way in which Mr. Everall helped them out of these difficulties. Mr. Macfarlane said that when the original structure was built many years ago half of its cost was met by the Punjab Government and half by the Kashmir Durbar ; but at the time of its reconstruction great difficulty was experienced with the Kashmir Durbar in the matter of their helping with contribution towards its cost. The Kashmir Durbar were very much against it for various reasons. One of them was that the Banihal Road had already been opened and the Kashmir Durbar, therefore, did not want to encourage traffic on other roads. Another reason was that the Abbottabad

road was also open, and therefore the Kashmir Durbar thought that it was quite possible for the people of the Punjab to come to Kashmir that way. Another reason was that the maintenance of the road between Kohala and Domel was a very expensive proposition, and the Kashmir Durbar felt that if they closed the Kohala road they would be saving the recurring cost of its maintenance. In other words, they practically refused having anything to do with its reconstruction. Further explaining the difficulties involving any correspondence through the Agent to the Governor-General, and the Government of India with the Indian States, he remarked that a great amount of correspondence went on for months and months and brought no satisfactory results. Their difficulties were also increased by the fact that the Kashmir Durbar was involved in very considerable difficulties owing to riots and so on. During this period transhipment had to take place from one side to the other and the actual toll which the Government was receiving fell to a considerable extent, because a very few annas or pies were charged. He further explained that it was only about this time last year that it was realized that Government, owing to the mid-stream girder being in a precarious condition, was faced with one of two alternatives, either to rebuild the span or to dismantle it altogether, and so cut off the whole of Kashmir from the Punjab by this particular road. Something, therefore, had to be done at once. The commercial interests in Rawalpindi and Murree were very much agitated over the whole state of affairs and they urged Government to rebuild the bridge. A very sporting offer was actually received from the contractors, one of whom was a well-known European contractor—and a well-known personality in the racing world, that he would rebuild the bridge free of cost if he were given the toll for one complete year. It was at this stage, the speaker remarked, that Mr. Everall came forward with his very ingenious proposition which the P. W. D. at once accepted.

Referring to the usual practice of the Engineers in exceeding their estimates, both in point of money and time, Mr. Macfarlane was glad to acknowledge that there was no excess beyond one or two per cent. in the construction of this work; and in regard to time, he explained that the work was completed within the stipulated period of 3½ months, having been started on the 15th of April and having been finished exactly on the 1st of August. Another interesting fact, he stated, was that immediately after the bridge was finished, the amount received by auction of the toll for the remainder of the year far exceeded the cost of the whole reconstruction.

Mr. S.L. Kumar in his remarks stated that on page 1, last para., the use of the word " elastic " for a soil said to be quick sand was unscientific. The world " yielding " or " unstable " would have been more appropriate.

Referring to page 2, in the last para., it appeared that the author had assumed that the main beam forms part of a rigid frame and its reactions inclined towards the centre line of the abutment. It was not understood why in the design the B.M. was figured at WL./8 as for a simply supported

beam. Even if the object had been to simplify calculations the B. M. should be taken as WL./12 for a fixed beam or at the most WL./10 for a partially fixed beam. It was desirable in reinforced concrete works of such magnitude to make a rational design with reasonable assumptions even though it might be a little more complicated as it would affect a saving of 20 °/₀ to 30°/₀. Another difficulty that had been introduced unnecessarily in the way of rational design was to have road girders supported on cantilevers on the sides of the ballast wall instead of cantilevers built up on the frame work on the sides of the main beams.

On Page 4, in the first two lines, it was stated that the portion of the original foundation retained after remodelling had been included in calculating the foundation pressure. It was not stated how the total load was distributed between the foundations of the original abutment and that of the new work.

On Plate I it was shewn that the maximum foundation pressure was 1·3 tons per sq. foot. In view of the fact that the foundation of the new work was not adequate for confining the quick sand, the pressure certainly appeared to be too high for soil described as quick sand.

Design No. II. Page 8.—The safe stress on stirrups had been taken as 10,000 lbs. per sq. inch. This was probably due to the assumption that the stirrups were subject to shear. Actually the stress in the stirrups was almost pure tension and the safe stress should have been taken at 16,000 lbs. per sq. inch.

Page 10. On this page, calculations were given for working out the stresses in the brickwork of the steining. It was not understood why the dead load amounting to 745 tons due to girders, road metalling, and trestle detailed on Page 6, should have been neglected while working out the stresses in the brickwork.

It was obvious that this load of 745 tons acting on the cantilevers must have its reaction somewhere. The reaction was provided by the top reinforcement which in its turn was anchored to the masonry of the well on the tilted side.

The reaction which was an upward force of 745 tons would produce a moment of the same sign as that due to the load of 1,190 tons, detailed on Page 10: taking the lower arm equal to about 4 feet, the moment due to this force about the centre $= 745 \times 4$.

The total compression in further calculations would be found to be 5·67 tons./sq. foot and the maximum tension to be 4·28 tons/sq. ft.

$$\frac{445}{646{\cdot}6} \pm \frac{1190 \times 7 \times 12{\cdot}5}{28423} \pm \frac{745 \times 4 \times 125}{28423}$$
$$= 0{\cdot}69 \pm 4{\cdot}98.$$

Mr. Trevor Jones remarked regarding the Kohala Bridge that the suspended span and the trestle were obviously rather vulnerable to heavy floods despite what had been done to protect them in the way of stone pitching and revetment work. Given that the hillside or bank on the British side could be reasonably secured from landslide, would it not be possible to span this gap of 46 feet by another cantilever anchored some way back into the bank. By such provision the Kohala Bridge might then regain the status of a permanent structure, a condition which could hardly be said to exist at the moment.

The author thanked the President and the members for the interest they had taken in his paper.

In reply to Colonel Macrae's remarks he stated that importance of selecting suitable materials for making sound concrete was of great value and he was glad to hear that Colonel Macrae confirmed his opinion regarding the poor quality of most of the sand and stone in Sind.

He wished to thank Colonel Macrae for the valuable advice he had given him in the development of an experimental type of road girder into a standardized type.

He said that regarding the term "elastic" to which Mr. Kumar had taken exception, it was ususal only to apply this term in a scientific sense in connection with the physical properties of iron, steel or alloys, and not to soils. The author was in agreement with him that "yielding" would be a better word to use than "elastic" in describing the soil at Kotri.

With regard to his criticism on the design of the main beam, Mr. Kumar would realise that the stresses in the completed R. C. work reached a maximum while it was subjected to D+L loads from the 100 feet span, and while the beam was still independent of the original masonry. In this condition, for purposes of calculating the B. M. at the central section of the main beam, the effective span was forty-five feet and not twenty-seven feet. However, the beam was only stressed appreciably during the working season, at a period of low water, when the soil had been proved to be capable of withstanding a definite horizontal thrust at the toe of the raft. The lines of thrust were drawn from the centre of the bases and passed through(0)in Fig. 3, at a lever arm distance of 27/2=13'-6" from the central section of the main beam. From this further explanation Mr. Kumar would be able to form the moment equations himself to verify the method used to proportion the main slab. It was only by making the most liberal assumption for the horizontal thrust consistent with safety that the author was able to obtain a reasonably proportioned main slab, the tension steel in which was stressed for a short time to well up to its safe value.

With regard to his criticism that the building of the R. C. cantilevers on the sides of the ballast wall introduces an unnecessary difficulty in the way of rational design, the author was unable to agree that his proposal

would in any way have cheapened, improved or in any particular facilitated execution of the work.

With regard to Mr. Kumar's criticism of the last two lines on Page 3, the completed structure was assumed to be monolithic for calculating foundation pressures. After a personal inspection at site during a period of high water it was decided to work to an average foundation pressure of 1·1 ton/square foot on the waterlogged sand 12 feet below ground level.

DESIGN NO. II, PAGE 8. Mr. Kumar in criticising the unit stresses adopted for calculating the vertical stirrups suggested that their action had not been understood by the designer of the well cap, and that they should have been calculated on a basis of 16,000 lbs. per square inch.

It was an acknowledged fact that cracks always appeared at sections of a loaded R. C. beam where tensile steel stresses approached 16,000 lbs. per square inch. These cracks had been proved to be harmless for most engineering structures; but the author assured Mr. Kumar that it was still not considered good practice by many ReinforcedConcrete Engineers to work stirrup stresses upto 16,000 lbs. per square inch. These high stirrup stresses initiated cracks in the planes where diagonal tension occurred and these cracks intersected the ordinary vertical cracks. Mr. Kumar was referred to an example in the Book " Reinforced Concrete " by Edward Ring in which an example was given for proportioning stirrups. 12,000 lbs. per square inch was a more usual figure for vertical stirrup stress but for this particular structure, it had been decided to limit the shear deflection and the plastic deflection and yield to as small a figure as possible because it had been impracticable on account of river conditions to keep the loaded cantilever unsupported by the brickwork for a sufficient number of weeks for the full deflection to develop.

With regard to Mr. Kumar's last criticism the author suggested that reference should be made to Page 5, No. II, Para. 3. The whole object of the R. C. work was to relieve the brickwork stresses at the underside of the cap and down to Section A B, shown on Plate 2. Below the Section A B the calculations made by the Chief Engineer, Surveys and Constructions Department, showed that the well brickwork was not overstressed by the eccentric superimposed loads.

The brickwork stresses found on Page 10, referred to a section immediately below the cap and did not, therefore, include the 745 tons referred to by Mr. Kumar which had already been taken by the R. C. cantilever, the nose of which had been unsupported when the 745 tons was applied. The cantilever dispersed this load into the well brickwork below Section A. B.

The author in his reply to Mr. Trevor Jones stated that the possibility would always have to be faced of another landslide taking place on the British side and the movement of a sliding suspended span catered for at the cantilevered nose.

Mr. Trevor Jones's suggestion to provide another cantilever anchored into the bank was certainly a sound one, and would perhaps enable the road engineer to re-establish communications after a normal landslide with more speed than if the present trestle abutment had to be re-established.

The President remarked that their hearty thanks were due to Mr. Everall for presenting such a valuable and interesting paper to the Congress. It was not unoften that peculiar situations arose during construction, and the examples given by Mr. Everall had a great educative value for those called upon to meet similar emergencies. He was particularly struck by the ingenious arrangement Mr. Everall had designed of combining a cantilever and a light suspended span on the Punjab side of the Kohala Bridge, with the result that a certain elasticity in the working of the bridge was introduced without straining the structure, in case any further forward movement took place. To his knowledge, this was the first example of such an ingenious design for a peculiar position.

As to Mr. Everall's portable standard steel road bridges, they reminded him of a certain feat of the Japanese Military Engineers in the crossing of the Yalu river which forms the frontier, between Korea and Manchuria. It was a Pontoon Bridge carried in sections on pack-mules. During the short span of a single night they erected the bridge, enabled the troops to cross the river, dismantled the bridge again, and took it on the same pack-mules into Manchurian territory. With Mr. Everall's portable sections, such a feat would now be possible to be performed by the Indian Army also.