

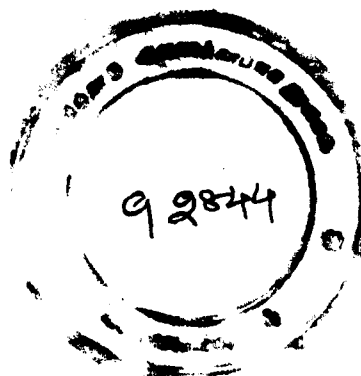
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Paper No. 164

"Testing to Destruction a 96-foot Prestressed Concrete Bridge Girder"

By

K. K. NAMBIAR AND P. V. RAJ

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SYNOPSIS

The Paper describes the test to destruction of a 96-ft 'U'-shaped prestressed concrete girder, 5 ft. 1 in. deep, prestressed with 256 high tensile steel wires (0.2 in. diameter) anchored on the Magnel-Blaton system. Four such prestressed girders with a 6 in. R. C. deck slab will constitute the superstructure of the completed bridge described in the Paper as designed for I.R.C. loading.

The test load was applied with four hydraulic jacks operating against cross-girders anchored down to buried counter-weights. The details of the design and construction of the prestressed girder, the method of test, the instruments used, the observations recorded, and an interpretation of the results are given. The first crack (visible with a magnifying glass) occurred when the applied load corresponded to a factor of safety of 2.0. The beam was taken to have failed under about six times the design live load. The possible effects of the continued application of the full working load over a long period are discussed.

1. INTRODUCTION

1.1. The first prestressed concrete highway bridge in India is now under construction over the Palar river near Chingleput on National Highway No. 45, about 40 miles from Madras.

1.2. When tenders, based on the official design, were called for the bridge, quotations based on several alternative designs were also received. The lowest tender was for a bridge with a prestressed concrete superstructure. As prestressed concrete was a comparatively new technique in this country the tender included a condition that one beam would be tested to destruction to demonstrate the behaviour of prestressed concrete under severe conditions of loading. After some modification of the original design, this lowest tender in prestressed concrete was accepted.

1.3. **The substructure.**—The bridge will be 2,202 ft long between faces of abutments and will consist of 23 spans, each 90 ft clear. The foundations under each pier consist of two 12-ft diameter circular wells with 8-ft dredge holes spaced 17 ft centre to centre and resting on hard moorum or soft rock. The average depth of foundations will be about 32 ft. The steining is of 1 : 3 : 6 cement concrete resting on a 1 : 2 : 4 R. C. curb. The wells are capped with 27 in. thick R. C. slabs to carry the massive piers. These piers are 6 ft wide at the top and 7 ft at the bottom (with a uniform batter) and have semi-circular cutwaters. The piers are 17 ft high and are of mass concrete (1 : 3 : 6) with 20 per cent plums of 8 in. size. The tops of the piers are finished with R. C. slabs to carry the precast prestressed girders.



Photo 2: Test beam under load. Beam on the right is ready for prestressing.

1.4. **The Superstructure.**—The decking consists of four precast, prestressed U-shaped girders spaced at 6 ft centres. Each girder is prestressed with eight cables, each consisting of 32 high tensile steel wires, 0.200 in. diameter. Four of these cables are straight and four parabolic. Six-inch thick internal diaphragms at 9 ft 6 in. centres are precast with the girders. Besides these internal diaphragms there are corresponding external diaphragms cast in situ, and provision is made for connecting these external diaphragms with the girders and with the deck slab. The internal diaphragms incidentally serve during construction to keep the cables in the desired position and both diaphragms serve to distribute the loads between the four girders.

1.5. After the precast, prestressed girders are placed on the span, the 6 in. thick R. C. deck slab is laid over them for their full width and length. Under the superimposed loads the deck slab and the prestressed U-section act as a composite beam. The kerbs, handrails and wearing coat are then cast. The details of the design of the superstructure are shown in Plate I.

2. DESIGN OF THE SUPERSTRUCTURE

2.1. Design Stresses

Type of stress	Working stress	Ultimate stress
Prestressed concrete, compressive stress in flexure	1500 lb./sq. in.	5000 lb./sq. in.
Tensile stress in flexure	Nil	Nil
Stress in high tensile steel	140,000 lb./sq. in.	220,000 to 240,000 lb./sq. in.
Assumed maximum losses due to shrinkage and creep in concrete and relaxation in steel	15 per cent	

2.2. Bending Moment and Stresses at Mid Span

2.2.1. Owing to the existence of the diaphragms, whose stiffness is many times that of the main girders, it is assumed that the deformation of all the longitudinal girders is

equal for any loading across the deck. Hence the bending moment due to live loads in any position on the deck is assumed to be equally shared by all the four girders.

2.2.2. Fig. 1 shows the section through one girder. The unshaded portion of the girder is the precast, prestressed unit. Its properties are :

Area of section	= 958 sq. in.
Moment of Inertia (I_{cg})	= 439,794 in. ⁴
Section Modulus, $Z_1 = I/Y_1$	= 14,000 in. ³
Section Modulus, $Z_2 = I/Y_2$	= 14,900 in. ³

After the deck slab is built, the composite section will act. The properties of the composite section are :

Area	= 1,390 sq. in.
Moment of Inertia, I_{cg}	= 793,794 in. ⁴
Section Modulus, $Z_1 = I/Y_1$	= 29,500 in. ³
Section Modulus, $Z_2 = I/Y_2$	= 19,800 in. ³

2.2.3. The weight of one precast girder including the internal diaphragms and cables is 1,192 lb per foot. The bending moment is equal to $1,192 \times 93^2 \times 1.5 = 15,400,000$ in. lb.

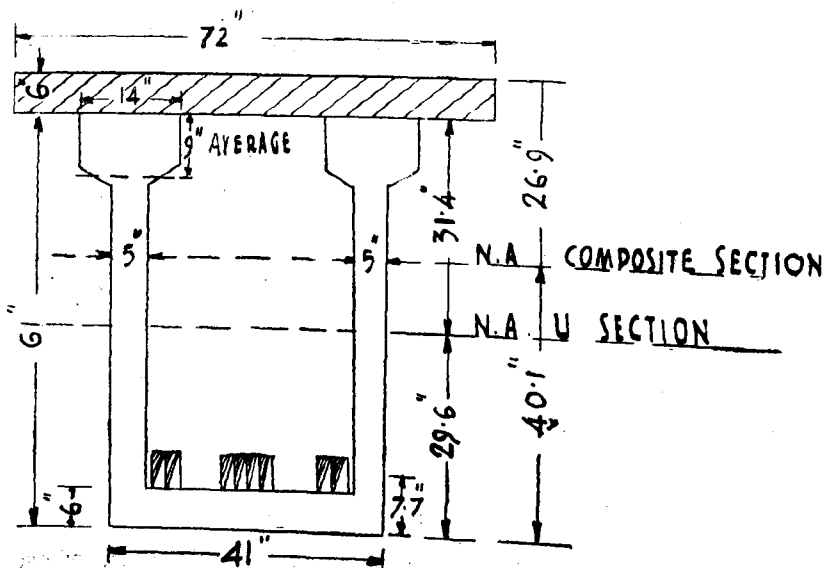


FIG. 1 SECTION OF THE GIRDER.

The weight of the deck slab and the external diaphragms is 2,088 lb per ft, i.e. 522 lb per ft per girder.

$$\begin{aligned} \text{Bending moment due to this} &= 522 \times 93^2 \times 1.5. \\ &= 6,800,000 \text{ in. lb.} \end{aligned}$$

2.2.4. The loads acting after the deck slab is set are :

- (1) Dead load due to the wearing coat, hand-rails and kerbs, = 1264 lb per ft
= 316 lb per ft per girder
giving a B.M. = $316 \times 93^2 \times 1.5$
= 4,100,000 in. lb.

- (2) Live load with impact per girder (2 lanes of class 'A' are severer than one lane of Class 'AA' loading in this case), giving a B.M. (Appendix I) = 12,200,000 in. lb.

$$\text{Total of (1) + (2)} = \underline{16,300,000 \text{ in. lb.}}$$

2.2.5. The prestressing force is applied through eight cables, each of 32 wires of 5 mm. diameter (0.2 inch) stressed to 140,000 lb per sq. in.

Four of these cables are straight and the other four are parabolic. These inclined cables are required because it is essential that the centre of gravity of the cables at the end of the girder should be within the kernel of the section to eliminate tension in the concrete. It is also desirable to distribute the anchorage over the face of the end block as uniformly as possible. The vertical component of the prestress in the inclined cables opposes the shear caused by live and dead loads and hence it is desirable that the vertical component of the prestress should be equal to the dead load shear plus half the live load shear. The arrangement of the cables as shown in Plate I is adopted as a reasonable compromise.

2.2.6. The total initial prestressing force is $8 \times 32 \times 0.0314 \times 140,000 = 11,25,500$ lb (where 0.0314 sq. in. is the cross-sectional area of each wire). The eccentricity of the cables at the centre of the girder = $29.6 - 7.7 = 21.9$ inches (See Fig. 1). Bending moment caused by eccentric prestressing force = $1,125,000 \times 21.9 = 24,600,000$ in. lb.

2.2.7. Stresses at Mid Span (See Fig. 2).

Stress when prestress is first applied lb/sq. in.	Stress after 15 per cent loss lb/sq. in.
Top Fibre	
Due to prestress =	
$+ \frac{1,125,000}{958} - \frac{24600000}{14000}$	
i.e. $+1175 - 1760$	
= -585	-495
Due to dead load = $\frac{15400000}{14000}$	
= $+1100$	$+1100$
Net $+515$	$+605$
Bottom Fibre	
Due to prestress =	
$+ \frac{1125000}{958} + \frac{24600000}{14900}$	
= $+1175 + 1650 = +2825$	$+2401$
Due to dead load =	
$\frac{1,54,00,000}{14,900} = -1030$	-1030
Net $+1795$	$+1371$

Stresses due to the deck slab and external diaphragms (vide para 2.2.3.):

$$\text{Top fibre } \frac{6800000}{14000} = +486 \text{ lb per sq. in.}$$

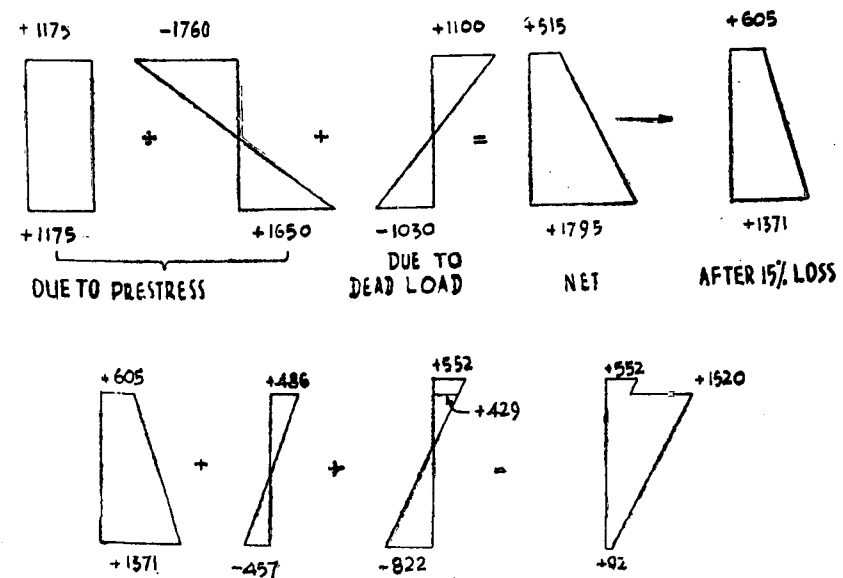
$$\text{Bottom fibre } \frac{6800000}{14900} = -457 \text{ lb per sq. in.}$$

Stresses due to all additional loads, i.e., wearing coat, kerbs, handrails and live loads (see para 2.2.4.):

$$\text{Top fibre } \frac{16300000}{29500} = +552 \text{ lb per sq. in.}$$

$$\text{Bottom fibre } \frac{16300000}{19800} = -822 \text{ lb per sq. in.}$$

The final stresses at mid span are shown in Fig. 2. At other points the dead and live load bending moments are smaller. The counter bending moment caused by prestressing is also smaller on account of the decreasing eccentricity of the parabolic cables.



FINAL STRESS DIAGRAM
FIG. 2 FIBRE STRESSES.

2.3. Shear Stresses near the Support

2.3.1. The maximum shear stresses occur in the web at the junction of the U-girder with the end block.

$$\text{Length between end blocks} = 85 \text{ ft}$$

Dead load

U-girder	1192 lb per ft
Slab, etc.	522 -do-
Kerb, handrails, wearing coat (see para 2.2.4.)	316 -do-
Total	2030 lb per ft

$$\text{Dead load shear} = \frac{2030 \times 85}{2} = 86,500 \text{ lb per girder (approx.)}$$

2.3.2. Live load shear

Class 'AA' is severer in this case.

Each track of the 70-ton tracked vehicle constituting the class AA load is directly carried by one girder. It is assumed that the fraction of the load coming over the deck on and beyond the first diaphragm is equally distributed over all the girders while the other portion of the load is directly carried by that particular girder.

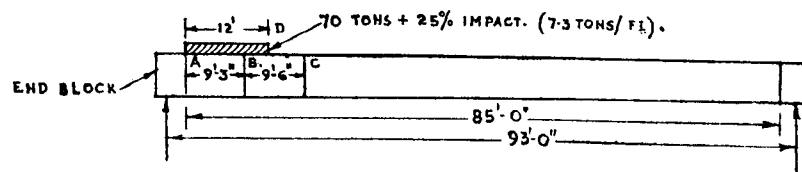


FIG. 3

Shear Near the end Block. For calculating reactions at A, B, and C the deck slab is assumed to be simply supported at the above points.

Reaction at A due to load AB

$$= \frac{7.3}{2} \times \frac{9.25}{2} = 16.9 \text{ tons.}$$

Reaction at B due to load AD

$$= \frac{7.3}{4} \times \frac{9.25}{2} + \frac{7.3}{4} \times 2.75 \times \frac{8.125}{9.5}$$

Shear at A due to reaction at B

$$= \left\{ \frac{7.3}{4} \times \frac{9.25}{2} + \frac{7.3}{4} \times 2.75 \times \frac{8.125}{9.5} \right\} \frac{79.75}{93} = 10.93 \text{ ton.}$$

Reaction at C due to load BD.

$$= \frac{7.3}{4} \times 2.75 \times \frac{1.375}{9.5}$$

Shear at A due to reaction at C

$$= \frac{7.3}{4} \times 2.75 \times \frac{1.375}{9.5} \times \frac{70.25}{93} = 0.57 \text{ tons.}$$

$$\left. \begin{array}{l} \text{Total Shear at A} \\ 16.9 + 10.93 + 0.57 \end{array} \right\} = 28.40 \text{ tons. or } 64,000 \text{ lb. (say).}$$

$$\text{Total shear} = 86,500 + 64,000 = 150,500 \text{ lbs.}$$

2.3.3. Negative shear due to prestress

The slope of the four bent cables in the end bay is 1 in 8. Therefore negative shearing force

$$= 0.85 \times \frac{1,125,000}{2} \times 1/8$$

$$= 59,700 \text{ lb} \quad (\text{see para 2.2.6})$$

$$\begin{aligned} \text{Therefore net shear force, } S &= 150,500 - 59,700 \\ &= 90,800 \text{ lb} \end{aligned}$$

$$\text{Shear stress} = \frac{S \cdot A \bar{y}}{b \cdot I}$$

$A \cdot \bar{y}$ above N. A. (See Fig. 1):

$$72 \times 6 \times 23.9 = 10,325 \text{ in.}^3$$

$$2 \times 14 \times 9 \times 16.4 = 4,133 \text{ in.}^3$$

$$2 \times 5 \times 11.9 \times 11.9/2 = 708 \text{ in.}^3$$

$$= 15,166 \text{ in.}^3$$

I, for the composite section = 793,794 in.⁴ (Vide para 2.2.2)

$$\therefore \text{Shear stress} = \frac{90800 \times 15166}{2 \times 5 \times 793794} = 174 \text{ lb per sq. in.}$$

The compressive stress at the neutral axis is 950 lb per sq. in.

$$\therefore \text{Principal tensile stress} = \frac{950}{2} - \sqrt{174^2 + \frac{950^2}{4}}$$

$$= 475 - 506$$

$$= 31 \text{ lb per sq. in.}$$

Shear stress at the junction of U-girder and deck slab is

$$= \frac{S \cdot A \bar{y}}{b \cdot I}$$

$$A \bar{y} = 72 \times 6 \times 23.9$$

$$= 10,325 \text{ in.}^3 \quad (\text{Vide Fig. 1})$$

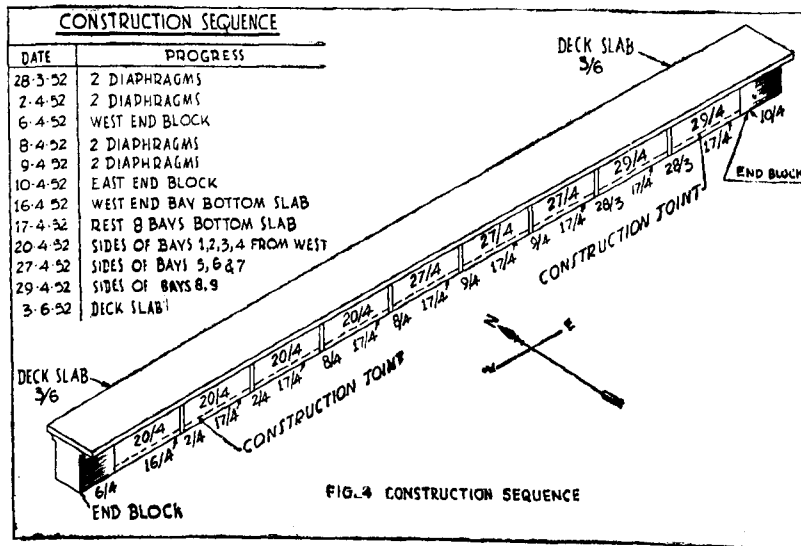
$\therefore$ Shear Stress

$$= \frac{90800 \times 10325}{2 \times 14 \times 793794}$$

$$= 42 \text{ lb per sq. in.}$$

3. CONSTRUCTION PROCEDURE

3.1. The precast prestressed unit was cast in stages. The internal diaphragms, 61 in. by 41 in. by 6 in., were cast separately with holes for cable ducts, and placed in position on the soffit shuttering. The solid end blocks were then cast. Thereafter the soffit slab was cast between the diaphragms together with the webs to a height of 4 inches. Finally the remaining portions of the webs were cast. (See Fig. 4)



3.2. This construction sequence meant that there was a horizontal construction joint in the webs four inches above the soffit slab. However, sufficient vertical stirrups were put in the webs to withstand the principal tension occurring at the neutral axis and consequently there was no danger of cracking at these joints.

3.3. The concrete mix used in this work was 1 : 1½ : 3 with a water-cement ratio varying from 0.42 to 0.45. The aggregate was not graded specially. All concrete was mixed in a pan-type mixer. In concreting diaphragms, webs, and end blocks, internal and external vibrators were used. For the soffit slab, only internal vibrators were employed. Ordinary portland cement was used for the precast units. The average strength of concrete at 28 days was 6,230 lb per sq. in. The order of casting the test girder with dates is shown in Fig. 4.

Rapid hardening cement was used in the deck slab which was cast later (see para 4.2.).

3.4. A camber of 1½ in. in the soffit slab was provided for at the time of casting.

3.5. Prestressing

3.5.1. Prestressing was done in a specified order to produce uniform stress in the girder and to prevent any jamming of loose wires. Appendix II gives the order of prestressing the cables, the load applied, and the elongation and slip for each pair of cable wires.

3.6. Pair loss

As there were 128 pairs to be stressed in succession, a loss of strain in the wires was expected to be caused by the elastic deformation of concrete and this was allowed for. This is known as "pair loss" and is computed as follows:

Maximum compressive stress in concrete at mid span (vide para 2.2.7.) 1795 lb per sq. in.

At the end of the span (vide para 2.2.7.) 1175 lb per sq. in.

∴ Average compressive stress in concrete (assume parabolic variation)

$$= 1175 + \frac{2}{3} \times (1795 - 1175) = 1588 \text{ lb per sq. in.}$$

∴ Average strain in concrete = $\frac{1588}{E_c}$ where E_c is the modulus of elasticity of concrete.

$$\text{Strain lost in the first pair of wires} = \frac{127}{128} \times \frac{1588}{E_c}$$

$$\therefore \text{Average strain lost in the wires} = \frac{1}{2} \times \frac{127}{128} \times \frac{1588}{E_c}$$

$$\text{So stress lost in the wires} = \frac{127}{256} \times \frac{1588}{E_c} \times E_s$$

(E_s being the modulus of elasticity for steel)

$$= \frac{127}{256} \times 1588 \times 7$$

$$= 5500 \text{ lb per sq. in.}$$

$$\therefore \text{Pair loss} = \frac{5500}{140000} = 4 \text{ per cent}$$

3.7. Slip

3.7.1. An allowance for a 1/16 in. slip in the wedges was also made. For stressing each pair of wires the actual extension of the wires as well as the load on the pressure gauge were checked and at the same time necessary corrections were made for the frictional losses in the jack.

3.7.2. In cable B₃ (see Appendix II) one sandwich plate (third from the top) was put in the wrong side outwards. This was noticed only when one pair of wires began slipping. The other three pairs of this sandwich plate were, however, held under full tension.

3.8. Deflection due to prestress

3.8.1. The ratio of dead load moment to that of prestressed counter-moment is :

$$\frac{15400000}{24600000} = 5/8 \text{ (say) — (vide paras 2.2.3 and 2.2.6.)}$$

That is, when 5/8th of the total wires are prestressed, the girder will "hog up" completely and during prestressing this was actually noticed to be the case. It was interesting to observe that during the first 5/8th part of the prestressing the girder did not deflect uniformly upwards. From Table 1 it is seen that there was positive as well as negative deflection up to this amount of prestress and after that there was only positive deflection. This indicates that the deflection was variable during prestressing till the girder hogged up completely. This meant that the full dead load moment was not available to reduce the effect of the eccentricity of the prestressing forces. We might also conclude that during this period the concrete might have been subjected to somewhat higher stresses than shown by calculation.

3.8.2. During the application of the prestress, various deformation measurements were taken so as to have a check on the calculated deformation.

Dial gauges were fixed to measure the vertical deflection of the beam at various places as shown in Fig. 5.

Table 1
Net Upward Deflection of the Test During
Girder prestressing

Pairs stressed in each cable	Deflection registered by gauge D ₆ (inch)	Deflection registered by gauge D ₇ (inch)
8 pairs in B ₄	+ .0034	+ .0029
4 pairs in B ₁	— .0171	— .0066
4 pairs in B ₁	+ .0096	+ .0062
8 pairs in T ₇	+ .0249	+ .0404
8 pairs in T ₆	— .0024	— .0066
8 pairs in B ₄	+ .0136	+ .0128
8 pairs in B ₁	+ .0208	+ .0252
8 pairs in T ₇	+ .0349	+ .0383
8 pairs in T ₆	— .0025	— .0053
8 pairs in T ₆	+ .0530	+ .0940
8 pairs in T ₆	+ .0030	+ .0030
8 pairs in B ₂	+ .0490	+ .0535
8 pairs in B ₃	+ .0805	+ .0750
7 pairs in B ₃	+ .0535	+ .0670
8 pairs in B ₂	+ .0345	+ .0505
8 pairs in T ₈	+ .0930	+ .1160
8 pairs in T ₈	+ .1160	+ .1740
1 pair in B ₃	+ .0195	+ .0050
Total	+ .5872	+ .7453
Corrected Total (0.1483)	+ .7355	+ .8936

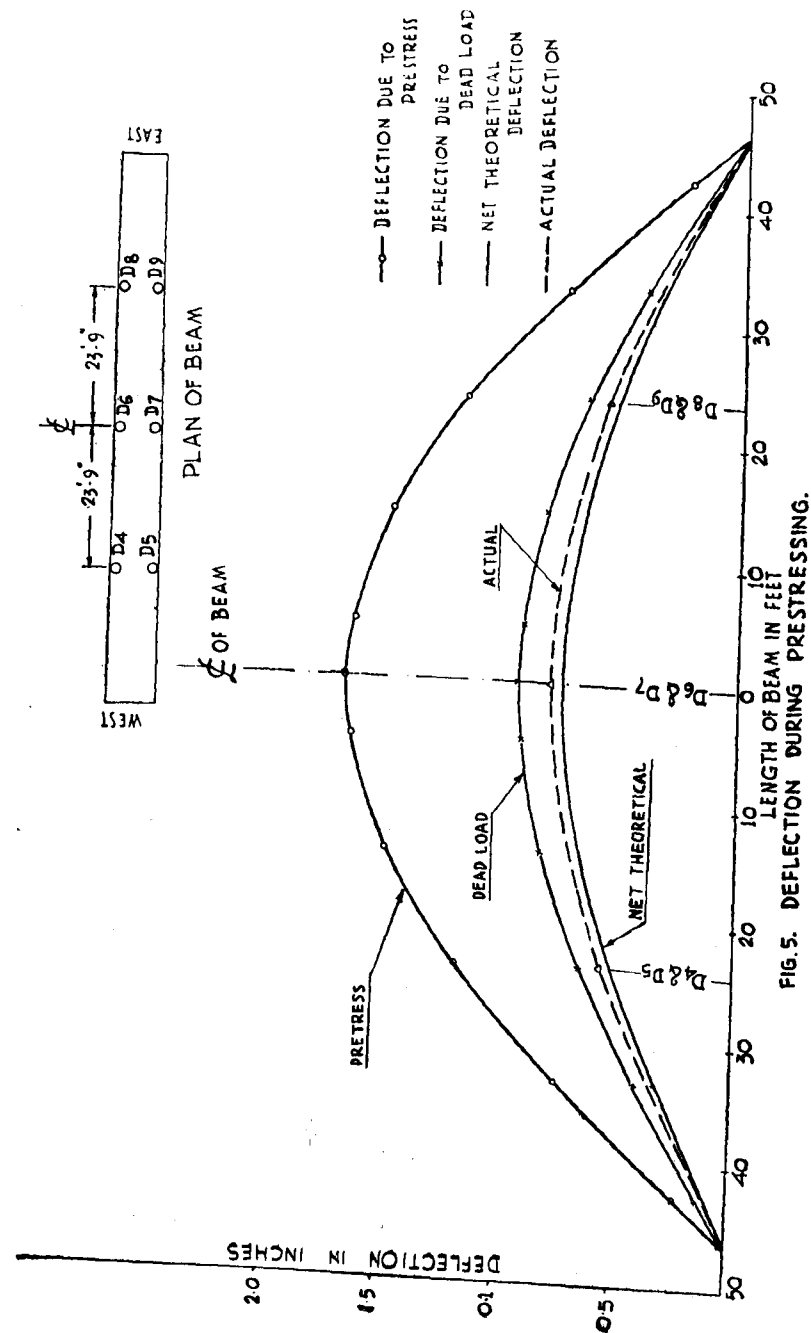


FIG. 5. DEFLECTION DURING PRESTRESSING.

3.8.3. Actual deflections.—The dial gauge readings for D6 and D7 during prestressing are given in Table 1. There is a marked difference between the total of D6 (+0.5872 in.) and that of D7 (+0.7453 in.). In the case of D4 and D5 the totals were +0.3900 in. and +0.3975 in. and for D8 and D9, +0.4347 in. and 0.4683 in. respectively. The ends of the beam rested on timber scantlings and the downward movements measured at the ends were 0.1170 in. at one support (east) and 0.1795 in. at the other (west). Hence,

the net actual upward deflection measured at

$$D4 - D5 = \frac{.3900 + .3975}{2} + .1640 = 0.5578 \text{ in.}$$

$$D6 - D7 = \frac{.5872 + .7453}{2} + .1483 = 0.8246 \text{ in.}$$

$$D8 - D9 = \frac{.4347 + .4683}{2} + .1323 = 0.5838 \text{ in.}$$

3.8.4. Calculated deflections:—Against these the calculated deflection due to the simultaneous action of the prestress and the dead load is:

$$\left. \begin{array}{l} \text{at } D4 - D5 \\ \text{and } D8 - D9 \end{array} \right\} = 0.6089 \text{ in.}$$

(This is actually at the third diaphragm from the centre).

$$D6 - D7 = 0.8546 \text{ in. (At the centre)}$$

These deflections were calculated with an assumed modulus of elasticity of concrete (E_c) equal to 3.9×10^6 lb. per sq. in. which was observed by testing a cube at 28 days, at about the time of prestressing. The actual modulus of elasticity is expected to be higher than that observed from a cube test. Moreover, for a concrete of 6,230 lb. per sq. in. strength this value of E_c seems to be rather low. Assuming a value of $E_c = 4.5 \times 10^6$ lb per sq. in.

the calculated deflection will be $0.6089 \times \frac{3.9}{4.5} = 0.5277$ in. and

$0.8546 \times \frac{3.9}{4.5} = 0.7407$ in. The actual deflections were about

10 per cent higher than the calculated deflections. This may be attributed to the deflections caused by shear stresses which were not considered in the calculated deflection. On the whole, the average observed values fairly conform to the calculated ones.

3.8.5. Fig. 5 shows the calculated and observed deflections during the application of prestress.

3.8.6. **End rotation** .—Two dial gauges were placed, one at the top and the other at the bottom, on each end face of the girder, and deformations were recorded. The observed end rotation of the girder was 0.00196 radian against the calculated rotation of 0.00150 radian.

3.8.7. **Elastic shortening** .—In order to measure the elastic shortening of the girder during prestress, marks were made on the sides near the ends of the girder and the end shortening was measured. The measured shortening was 0.39 in. against the calculated shortening of 0.295 in.

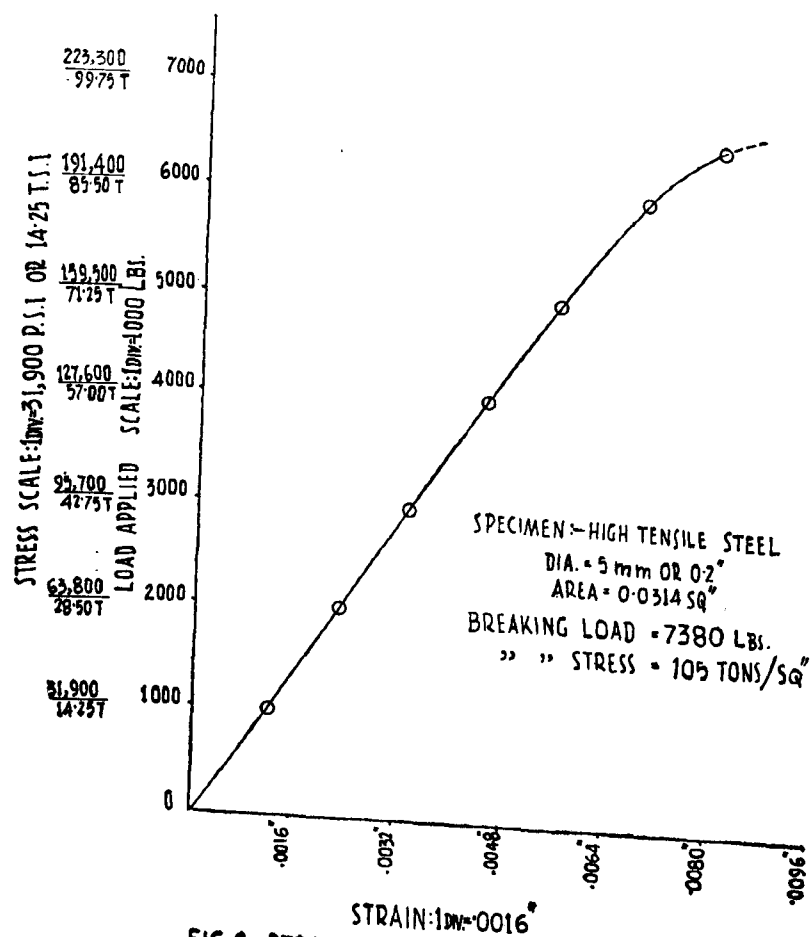


FIG. 6. STRESS-STRAIN CURVE OF HIGH TENSILE STEEL.

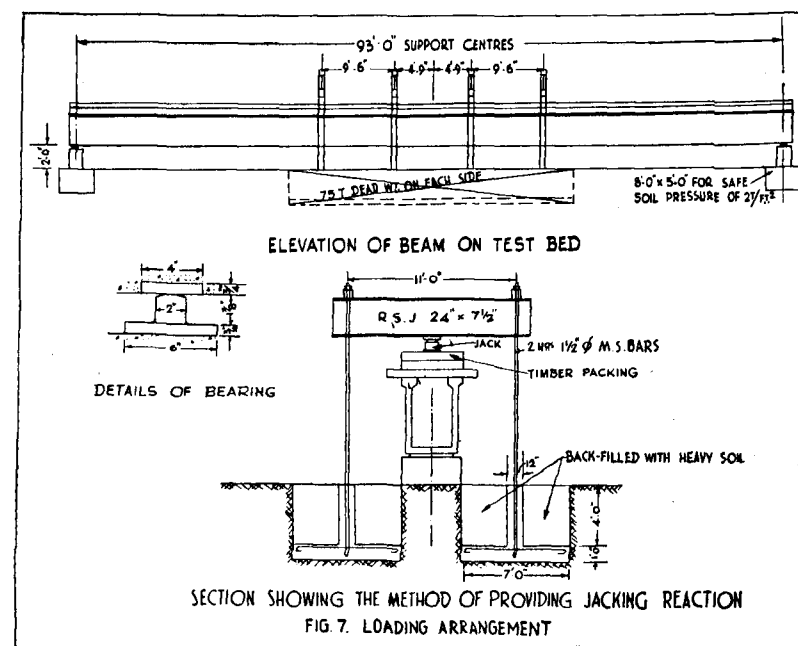
3.8.8. **Strain in concrete** .—Electrical strain gauges (S. R. 4) were used to measure the strain in concrete at mid-span and at the ends. The observed readings for the first half of the prestress were very erratic. At that stage the instrument went out of order and further readings could not be taken.

3.9. The stress-strain curve of the high tensile steel is given in Fig. 6.

4. THE LOAD TEST

4.1. After prestressing, the girder was lifted up two feet and placed on specially prepared abutments. Bearings of steel were so placed as to give an effective span of 93 ft (See Fig. 7).

4.2. All the cables except two were grouted with cement mortar. To test the efficiency of the anchorages more severely these two cables, T7 and T8 (vide App. II), were left free. The R.C. deck slab, 6 ft wide and 6 in. thick, was then laid to the full length, in 1 : 2 : 4 concrete with rapid hardening cement. At the time of testing the girder, the strength of this deck concrete exceeded, 6,000 lb. per sq. in.



4.3. At the time of the test it was assumed that only 10 per cent loss in prestress had taken place and so the stresses at midspan of the U-section (with the deck slab) were:

$$\begin{aligned} \text{Top fibre} &= 0.9(1175 - 1760) + 1100 + 486 \\ &= +1060 \text{ lb. per sq. in.} \end{aligned}$$

$$\begin{aligned} \text{Bottom fibre} &= 0.9(1175 + 1650) - 1030 - 457 \\ &= +1056 \text{ lb. per sq. in.} \end{aligned}$$

The whole precast section was thus under a uniform stress of about 1,060 lb. per sq. in. while the deck slab was not at all stressed.

4.4. Loading arrangement

4.4.1. The girder was loaded at the four central diaphragms, at 9 ft 6 in. centres, by means of hydraulic reaction jacks placed on timber packing on the deck slab and acting against 24 in. $\times$ 7 $\frac{1}{2}$ in. rolled steel joists. These joists were anchored down by means

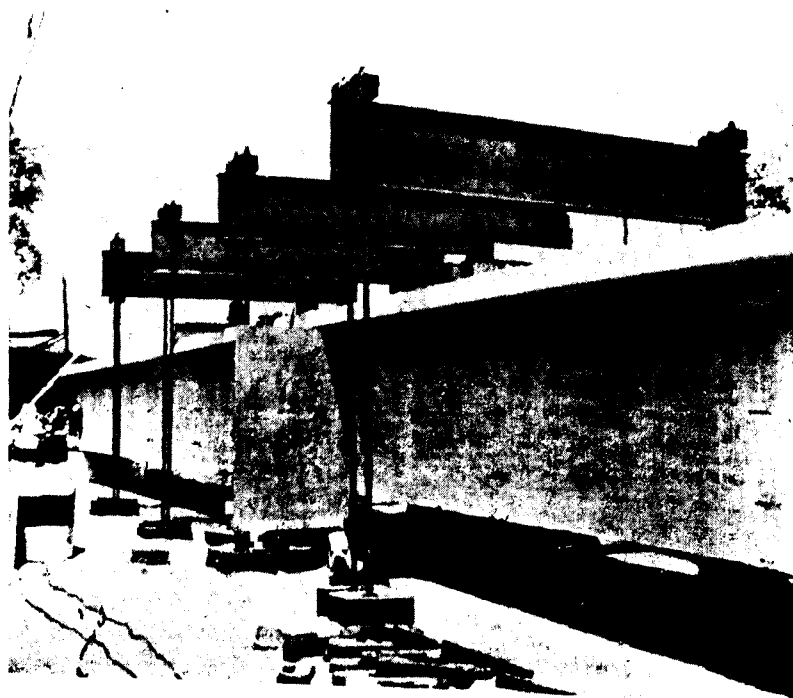


Photo 2: Close up of the Loading Arrangement.

of 1 $\frac{1}{4}$ in. diameter mild steel bars to two inverted concrete T-beams buried in the ground (Fig. 7).

4.4.2. Since in actual use the section would be subjected to maximum stress at mid-span, the four equal jack loads were adjusted to give the requisite bending moment at mid-span.

4.4.3. If W lb. were the load on each jack, the bending moment at mid-span would be $888 W$ in. lb.

$\therefore W$, equivalent to the wearing coat, handrails, and kerbs together with the external diaphragms which were not actually cast on the test girder,

$$= \frac{5115000}{888} = 5760 \text{ lb} = 2.57 \text{ tons.}$$

$$\text{Load on four jacks} = 4 \times 2.57 = 10.28 \text{ tons}$$

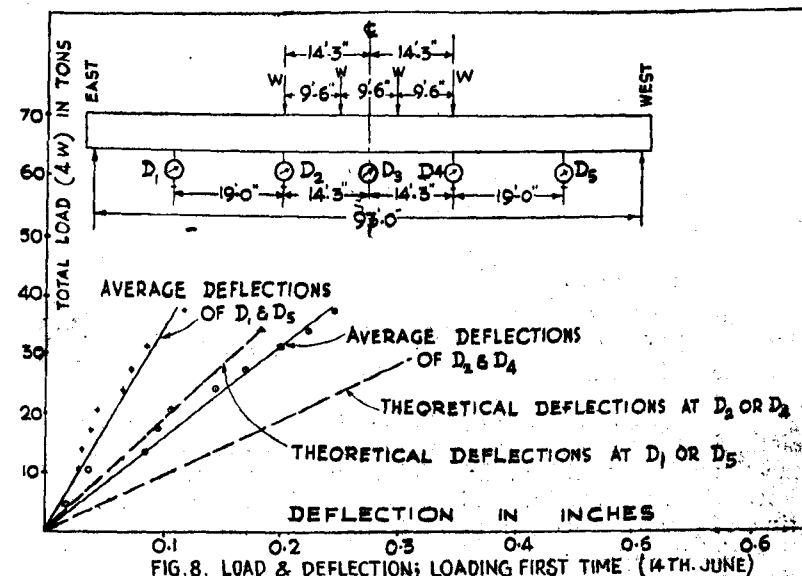


FIG. 8. LOAD & DEFLECTION; LOADING FIRST TIME (14TH. JUNE)

W , equivalent to full live load (vide paragraph 2.2.4.)

$$= \frac{12209000}{888} = 13,750 \text{ lb or } 6.14 \text{ tons.}$$

$$\text{Load on four jacks} = 4 \times 6.14 = 24.56 \text{ tons.}$$

$$\text{Full working load} = 34.84 \text{ tons.}$$

4.5. Instrumentation

4.5.1. Various measurements were taken during the loading to have a comparison of the actual with the calculated deformation. Dial gauges, D_1 , D_2 , D_3 , D_4 and D_5 , as shown in the Fig. 8, were fixed to measure the vertical deflection of the girder.

4.5.2. To measure the end rotation of the girder, two clinometers, C_1 and C_2 , were fixed on the top of the deck slab at the ends. As an additional check on this end rotation, two dial gauges were fixed at each end face of the girder ($D_6 - D_7$ and $D_8 - D_9$ in Fig. 9).

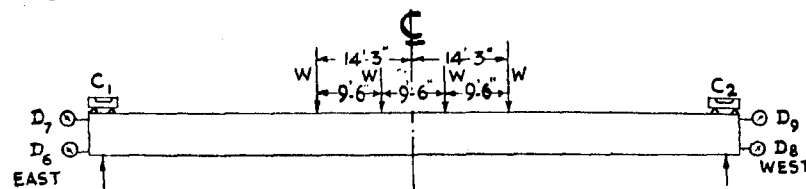


FIG. 9. MEASUREMENT OF END ROTATION

4.5.3. To measure actual strain in concrete, electrical strain gauges (Type S.R 4) were fixed to the concrete surface at mid-span at positions shown in Fig. 10.

4.5.4. Three optical extensometers were fixed on one side but, owing to a high wind, consistent results could not be obtained.

4.6. Load Application

4.6.1. The test girder was loaded on two days, viz., the 14th and 15th June 1952. The first day the total load applied was the full working load of 34.84 tons.

4.6.2. Fig. 11 shows the observed and calculated deflections along the length of the girder under the full working load. Here the Young's modulus of Elasticity for concrete was assumed as 6×10^6 lb. per sq. in. against 4.5×10^6 lb. per sq. in. at the time of prestress (see paragraph 3.8.4). The reasons for adopting this higher value were :

- the concrete was older and therefore stronger than at the time of prestress ;
- the effect of applying a live load to a composite beam with a top flange (not prestressed) has mainly a tendency to reduce the stresses in the prestressed portion and the modulus of elasticity of concrete for release of stress is greater than that for increase of compressive

stress. Also the stresses in the compressive region, i.e., the *in situ* top flange were very low and the modulus of elasticity in compression for concrete is greater at lower stresses than at higher stresses.

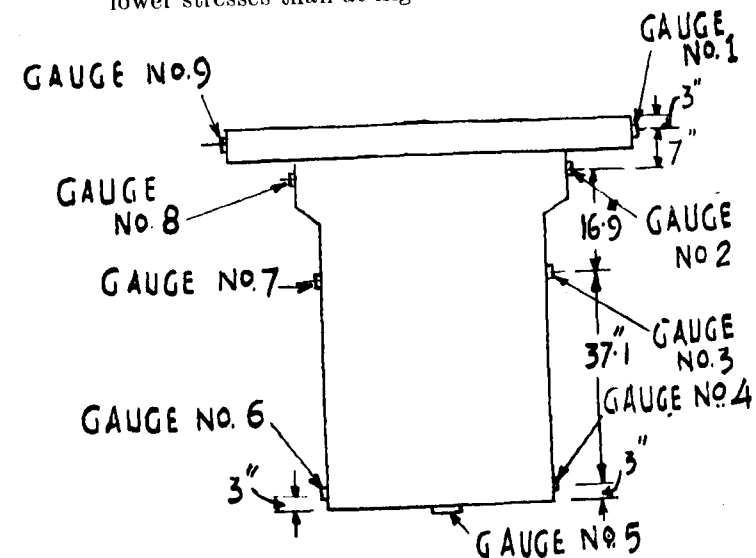
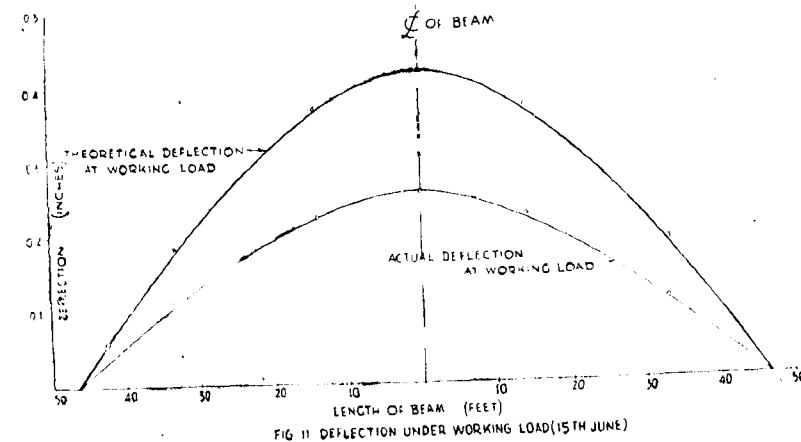


FIG. 10. DISPOSITION OF SR 4 GAUGES

4.6.3. The observed end rotation of the girder at this stage was 0.0006 radian against the calculated figure of 0.0011 radian. [Fig. 15.]



4.6.4. Figures 8, 11, and 12 show the deflection of the girder on 14-6-1952 and 15-6-1952. The centre dial gauge, D_3 , got stuck up on the first day and gave rather erratic readings. It was changed on 15-6-1952. Attempts to observe the permanent set over 20 hours of loading proved unsuccessful. As the jacks were hydraulic, pumping was continued up to 11 p.m. on the 14th June but next morning two jacks were found to be leaking, and hence the test girder could not be taken to have been kept loaded fully. This is a general defect in the use of hydraulic jacks for test loading over long periods.

4.6.5. The load deflection curve (Fig. 12) was a straight line up to the first crack. To obtain a zero stress in the bottom fibre the load required on each jack would be :

$$\frac{1056 \times 19800}{888} = 23,580 \text{ lb or } 10.5 \text{ tons}$$

Here, if the high tensile steel were also assumed to take part in resisting the bending moment, the section modulus for the bottom fibre would be increased by 10 per cent or so. The load on each jack would then be $1.1 \times 10.5 = 11.5$ tons and the load on all the four jacks would equal 46 tons.



Photo 8: A View of Deflected Beam

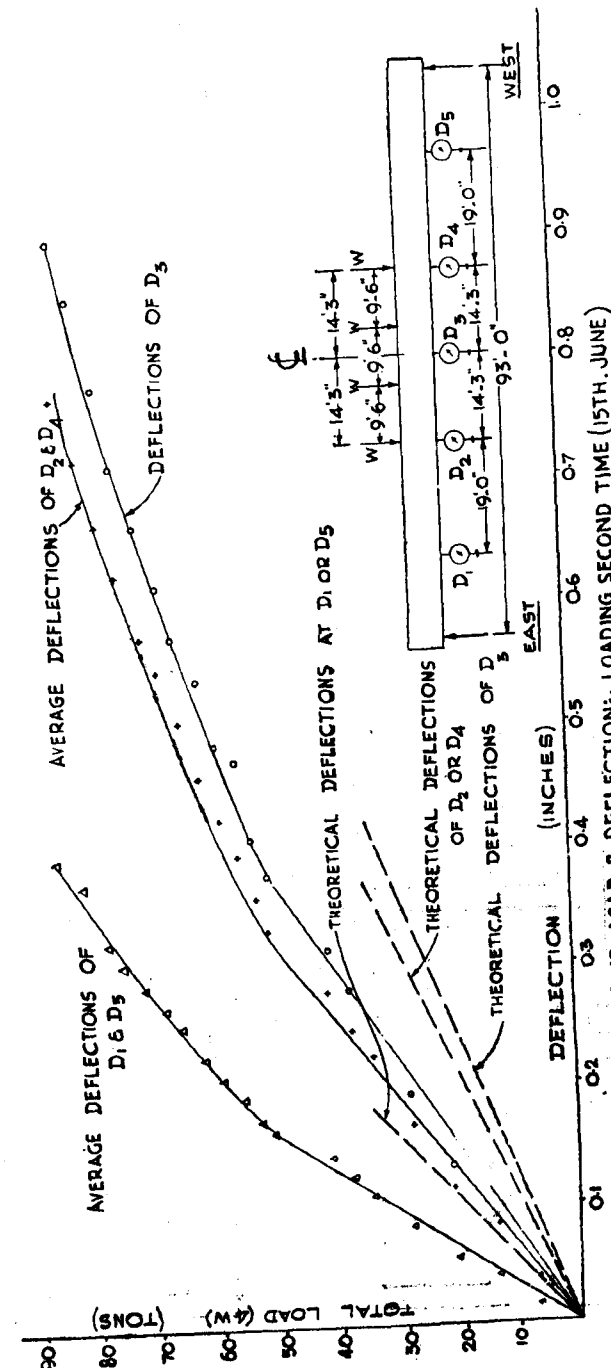


FIG. 12. LOAD & DEFLECTION: LOADING SECOND TIME (15TH JUNE)

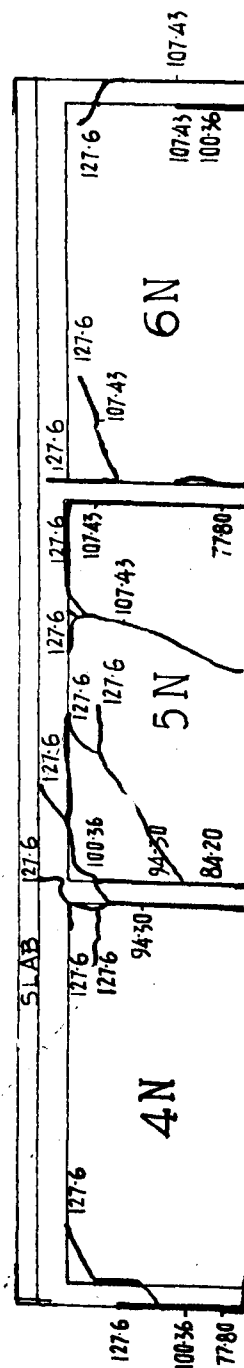
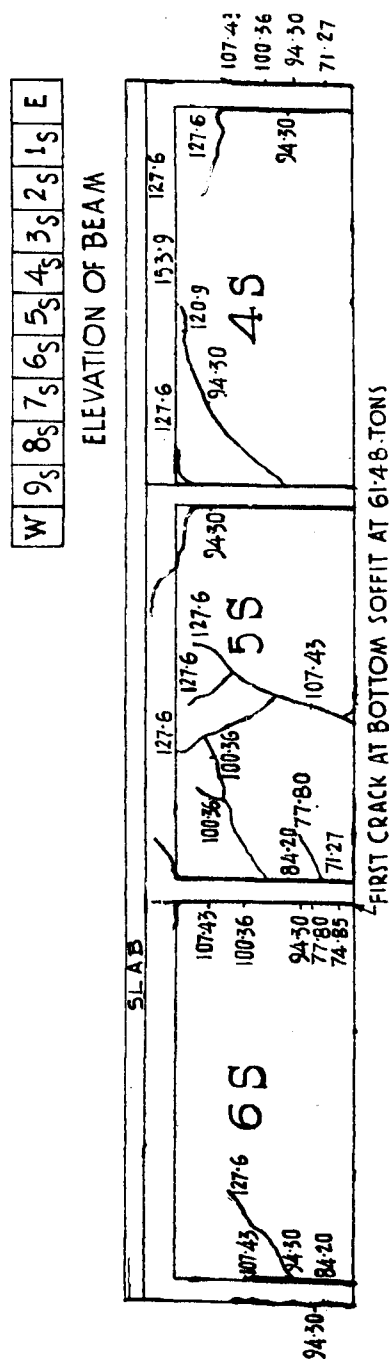


FIG. 13. CRACK PATTERNS

Figures show the load (tons) at which the cracks appeared.



Photo 4: View showing the Cracks at the Centre of the Beam on the South side.

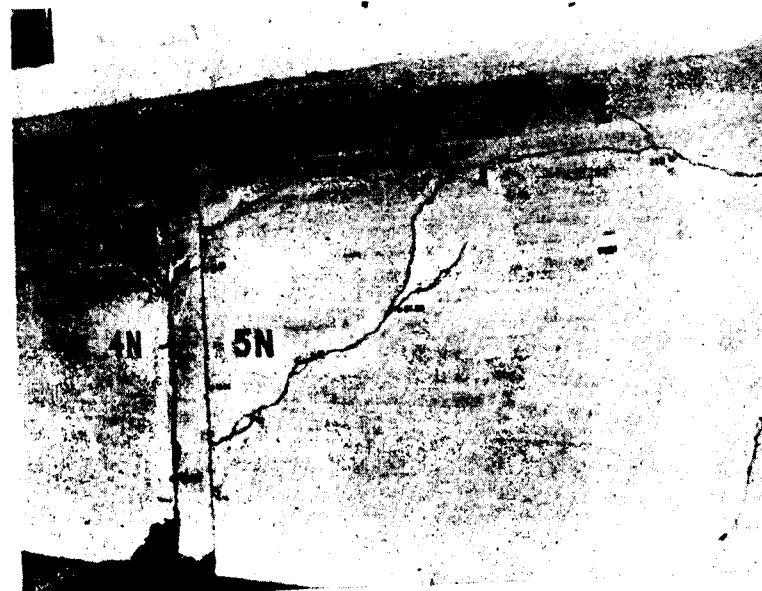


Photo 5: View Showing the Cracks on the North Face of the Beam.

4.6.6. **Factor of safety.** The first visible crack appeared at 61 tons. Hence the flexural strength of concrete (which was laid against the smooth surface of the diaphragm), would be

$$= f = \frac{(61 - 46)}{4} \times \frac{2240 \times 888}{1.1 \times 19800} = 410 \text{ lb per sq. in.}$$

The factor of safety against the first visible crack

$$= \frac{61.0 - 10.28}{24.56} = 2.0$$

From the graphs of load against deflection (Fig. 12), it is apparent that the concrete must have actually (but invisibly) cracked at a load of about 52 tons, i.e., at the change of slope of the deflection curves. Hence the true flexural strength of the concrete

$$= \frac{(52 - 46)}{4} \times \frac{2240 \times 888}{1.1 \times 19800} = 150 \text{ lb per sq. in.}$$

Hence the factor of safety against cracking = $\frac{52 - 10.28}{24.56}$
= 1.70

4.6.7. **Crack patterns.** After the first crack the load-deflection curve is not a straight line. As the cracks extend upwards the section modulus changes and so do the deformations. Fig. 13 shows the cracks in the girder at various loads.

Between the two central load points there is no external shear acting and hence the cracks here should have been all vertical. But when the load reached 90 tons some cracks started curving. A probable explanation is that, with the appearance of the cracks at the diaphragms, the U-section completely separated and produced tension in the vertical direction due to its own weight. Another explanation is that, as the cracks extend upwards, there is a large reduction in the effective sectional area of the uncracked portion of the U-section. The prestress, however remains constant (or, if anything, slightly increases as a result of deflection). The compressive stress in the uncracked portion of the section therefore increases rapidly and this results in a tensile stress developing in the vertical direction. The intensity of this tensile stress depends on the value of Poisson's ratio for the concrete.



Photo 6: Another View Showing Cracks on the South Face of the Beam.

4.6.8. It is also interesting to note that the central crack appeared even after the girder had cracked at the two adjacent diaphragms. This points to the presence of an effective bond between the wires and the grout and the soffit concrete in this section.

4.6.9. The strains measured with SR4 gauges No. 4 and No. 6 (Fig. 10) were converted into stresses and are shown in the Fig. 14. The corresponding stresses were found by assuming $E_c = 6 \times 10^6$ lb per sq. in. The results are rather erratic though at some loads these stresses nearly conform to calculated stresses. The strain measurements by SR4 gauges, though reliable and accurate under

laboratory conditions, are of limited scope under varying outdoor weather conditions. The readings of the other SR4 gauges (viz. those at S_1 , S_2 , S_3 , S_6 , S_7 , S_8 and S_9) were of a similar pattern.

Measurements were taken till the total load was 96 tons and thereafter all gauges were removed.

4.6.10. The first crack at 61 tons could be observed only with a magnifying glass. When easily visible cracks at 75 tons were observed the load was released and all cracks closed fully.

The load was again applied up to 96 tons and released to see how these wide cracks closed, and all cracks closed remarkably.

Finally the load was applied up to 143.5 tons and the girder deflected as much as about 10 inches. As the deflections were large and it was dark by this time it was thought that the girder was nearing failure. Later when the load was released the girder was found to be still elastic.

4.6.11. Three days later an attempt was made to break the beam and the load was increased up to 158 tons with a deflection of about 15 inches. At this stage one of the hydraulic jacks slipped suddenly and the girder came back almost to its normal position. The cracks, which were as wide as $1\frac{1}{2}$ in. under this load, all closed again. No further attempt was made to destroy the girder.

4.7. How a prestressed concrete girder fails

4.7.1. A prestressed concrete structure fails either by the crushing of the concrete or by the tension failure of the steel. When the quantity of steel is excessive, the concrete in compression fails by crushing suddenly without showing any tension cracks. Such a failure will come about without any warning and is not desirable. When the quantity of steel is very small, a tension failure in steel will result and this happens sometimes suddenly when there is transference of load from concrete in tension to the steel. When a well balanced quantity of steel is provided, the concrete on the tension side cracks and large deformation takes place, thus giving a good warning.

4.7.2. As cracks develop, the net area in compression decreases and thereby increases the compressive stress in concrete up to crushing. These heavy cracks result in a large deformation. As the load increases this large deformation causes a proportionate increase in the stress in steel. As a result of this the permanent set in the steel also increases and

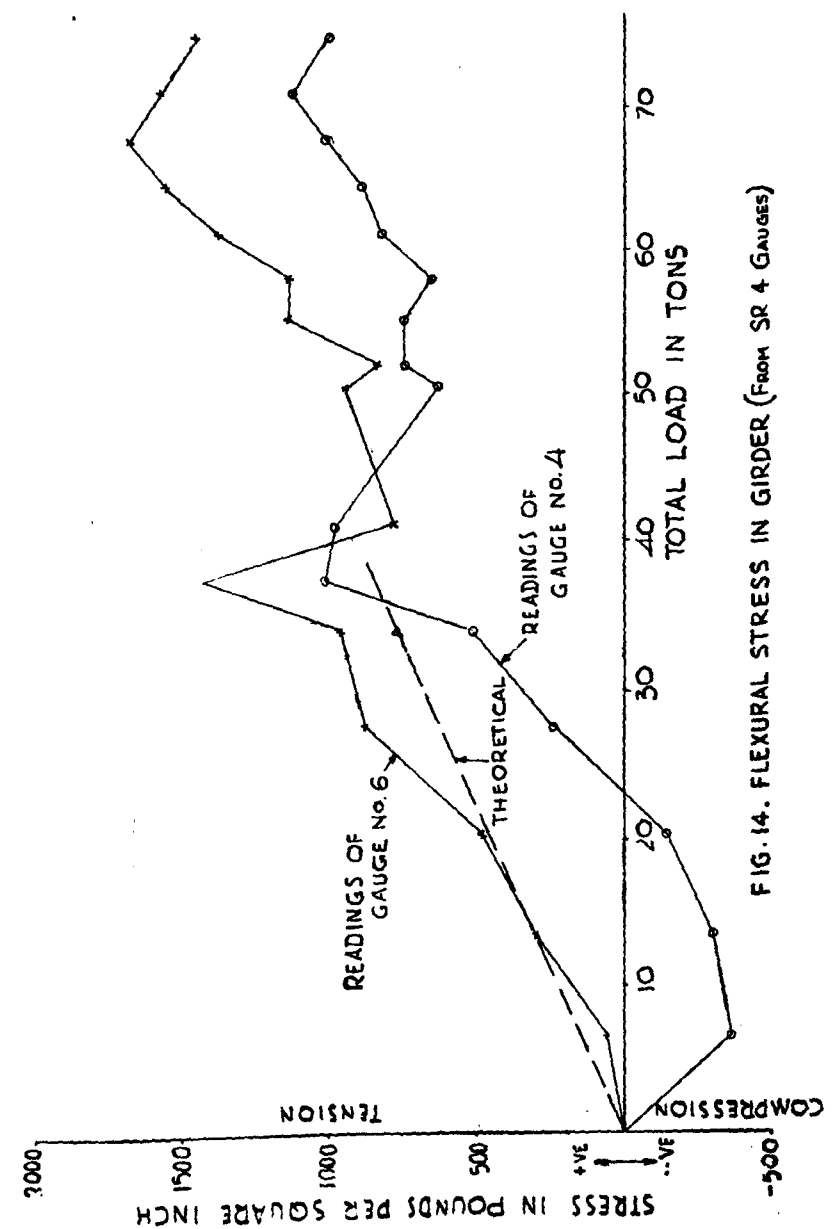


FIG. 14. FLEXURAL STRESS IN GIRDER (FROM SR 4 GAUGES)

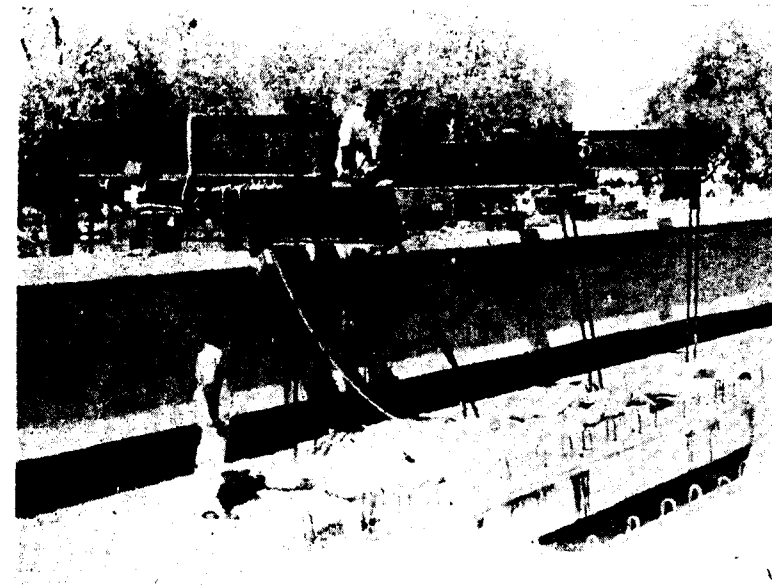
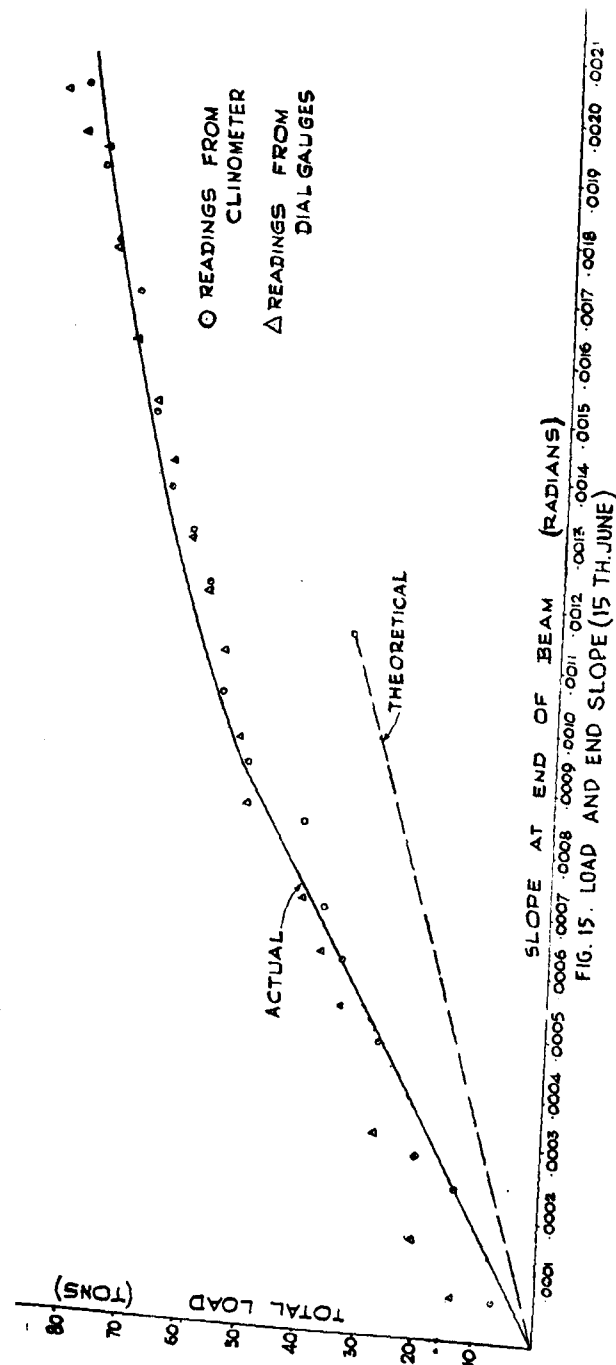


Photo 7: Photo shows the slipping of Jacks when the Load was Increased to 158 Tons; the Beam has Regained its Original Position

thereby decreases the prestress and elastic properties of the girder. When the steel stress is such that the permanent set at that stress is equal to the elastic deformation at the working stress, the whole prestress is lost and the beam behaves just like any ordinary reinforced concrete structure. So the characteristics of the stress-strain curve of the high tensile steel play an important role in the ultimate strength of the prestressed concrete structure.

4.7.3. The type of steel used on the Palar bridge has a low permanent set even near to the ultimate stress. The strain at the working stress is 0.42 per cent and the stress in steel to have this 0.42 per cent permanent set is above 97 tons per sq. in. as determined in the laboratory.

4.7.4. To reach this 97 tons per sq. in. in the test girder it should deflect roughly as calculated below:

The stress (i.e., prestress)
in steel without any load
is about 56 tons per sq. in.

The increase in stress, $97 - 56 = 41$ tons per sq. in.
This gives a total elongation

$$= \frac{41 \times 2240}{30 \times 10^6 \times 96 \times 12} = 3.52 \text{ inches.}$$

For this elongation the deflection of the girder would be :

$$\sqrt{\left(\frac{96 \times 12 + 3.52}{2}\right)^2 - \left(\frac{96 \times 12}{2}\right)^2} = 45 \text{ in.}$$

But in attaining a deflection of 15 in. the girder actually cracked up to the deck slab. It is therefore likely to fail before the large deflection of 45 in. is reached.

The resisting moment of concrete is

$$72 \times 6 \times \frac{2}{3} \times 6000 \times 56 = 96,768,000 \text{ in. lb.}$$

$$\text{This is equivalent to } = \frac{96,768,000}{2240 \times 888}$$

$$= 48.5 \text{ tons at each loading point or a total load of 194 tons.*}$$

(The factor $\frac{2}{3}$ for the compressive stress is taken assuming parabolic variation of the stress strain curve in concrete and also this $\frac{2}{3} \times 6000 \times \frac{105 \times 2240 \times 8}{72 \times 6}$

The resisting moment of steel at failure is $105 \times 2240 \times 8 \times 56 = 105,369,600$ in. lb, the area of steel in each of the 8 cables being one sq. inch and the ultimate strength 105 tons per sq. in.

From this ultimate resisting moment, if we deduct the moments due to dead weight of the girder (vide para 2.2.3) and *in situ* deck slab, the moment due to superimposed dead and live loads at failure would be :

$$= 105,369,600 - 15,400,000 = 89,969,600$$

$$= 84,184,600 \text{ in. lb}$$

This represents a load of $\frac{84,184,600}{888 \times 2240}$ tons or 42.3 tons at each of the four loading points, or a total load of 170 tons.* Hence the girder would fail at the lower of the two values, viz., 170

* For the factor '888' see para 4.4.3.

tons. With a slight increase of the load above 158 tons the cracks would go into the slab and reduce the resisting moment of concrete so that the girder is expected to fail finally by the crushing of concrete.

5. CONCLUSIONS

5.1. In ordinary reinforced concrete, the working stresses will give fair indication of the factor of safety of the structure but in prestressed concrete this is not the case. The factor of safety against failure has to be determined in each case.

5.2. The test showed that the design of the prestressed concrete girder was adequate. The structure gave a factor of safety of 2.0 at first crack and more than six against failure considering the live load only. The factor of safety against failure considering all loads is :

$$\frac{158 + 42.57}{42.57 + 10.28 + 24.56} = \frac{200.57}{77.41} = 2.60$$

(42.57 tons is the equivalent weight of U-beam and deck slab)

This is more than ample. The German regulation for prestressed concrete specifies, in fact, a factor of safety of 1.75.

5.3. As pointed out (in para 4.7.1) it is important that a prestressed concrete girder should give ample warning of failure. Since the beam showed readily visible cracks at 75 tons load which is less than half the ultimate strength of the beam, the design fulfils this requirement.

5.4. Observations during the test revealed that there was no slip in the anchorages. This showed that the Magnel-Blaton system of anchorages is effective.

5.5. Even with a load six times the live load, or 2.6 times the dead and live load taken together, there was no sign of shear cracks near the support though this stress was of the order of 800 lb. per sq. in. In ordinary reinforced concrete a similar beam would have shown great distress due to shear.

5.6. The concrete in the end blocks which was subjected to severe concentration of loading under the distribution plates showed no signs of failure. The size of the standard Magnel-Blaton distribution plates was, therefore, adequate to spread the load safely on the concrete.

5.7. In this Paper mention has been made of the failure of hydraulic jacks (para 4.6.4.) to maintain the full load for any length

of time and the failure of the SR4 electrical strain gauges (para 3.8.8) and the optical extensometers (para 4.5.4.) to give accurate readings when used outside the laboratory. It is recommended that, if possible, such load tests should be made with weights applied to the structure under test either directly or indirectly but not through a hydraulic system.

For measuring strains in concrete it is suggested that metal studs or bars buried in the concrete in selected critical positions should be used. The distance between marks on adjacent studs may be measured before and after loading with an extensometer, Metzgar gauge, or other similar instrument. This method should give more reliable results than those obtained in this test.

5.8. While the test results, compared with theoretical expectations, have demonstrated that the method of applying the prestress and the anchorage system are efficient, the question arises how far this prestressing force is permanent.

Other factors on which the durability of prestress depends are the creep and shrinkage of the concrete and the relaxation of the high tensile steel reinforcement. The major proportion of the losses, for which an allowance of 15 per cent is usually made, is due to shrinkage and creep of the concrete. Research over the past thirty years has provided sufficient confirmation of theory in this regard.

As far as relaxation of the steel is concerned, this is a small proportion of the total losses provided that the correct quality of steel at the recommended stresses is used. Measurements conducted on actual prestressed concrete structures confirm laboratory results obtained by steel manufacturers and independent research workers over a long period of years.

5.9. Prestressed concrete structures are expected to have a longer life than reinforced concrete structures because of the increased resistance to corrosion offered by crack-free concrete.

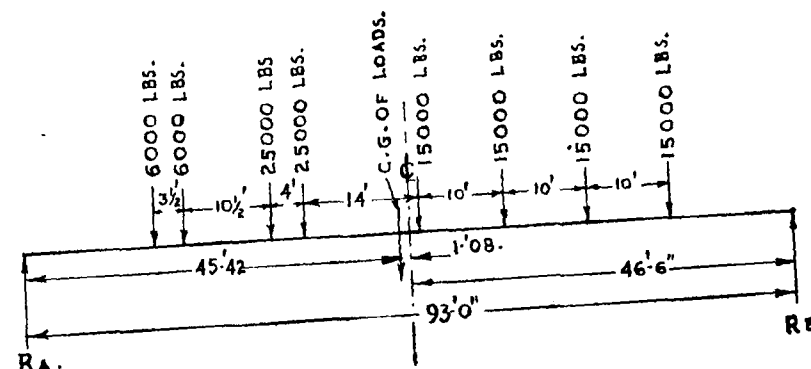
5.10. The economy which results from the use of prestressed concrete construction would be enhanced if the special materials which are at present imported could be manufactured in this country.

5.11. The Authors wish to congratulate Messrs. The Associated Engineering Syndicate and their associate contractors, Messrs. Rayalaseema Constructions (licensees in India for the Magnel-Blaton System) for their courage and enterprise in testing the very first girder made by them before a gathering of engineers from all over India.

Appendix I

LIVE LOAD BENDING MOMENT (See para 2.2.4.)

Maximum bending moment due to live load will occur when two lanes of class A loading are placed on the span with the centre of the span midway between the centre of gravity of the loads and the nearest heavy load. This arrangement is shown in the sketch.



$$R_b = \frac{1,22,000 \times 45.42}{93} = 59,600 \text{ lb}$$

Maximum bending moment occurs under the 15,000 lb load nearest the centre and is equal to

$$59,600 \times 45.42 - 15,000 (30 + 20 + 10) = 18,07,000 \text{ ft lb.}$$

The impact factor for class A loading

$$= \frac{15}{20 + \text{span length}} = \frac{15}{20 + 93} = 13 \text{ per cent}$$

$$\text{Bending moment for two lanes} = 18,07,000 \times 2 \times 1.13$$

including impact.

$$= 40,83,000 \text{ ft lb}$$

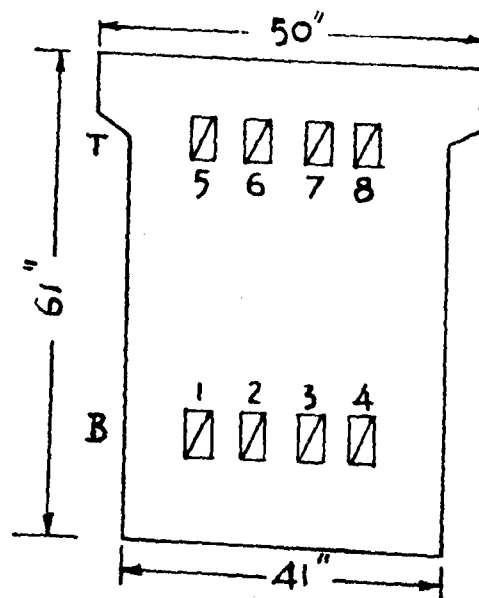
Bending moment per beam.

$$= \frac{40,83,000}{4} \times 12 = 12,200,000 \text{ in lb}$$

APPENDIX II

ORDER OF STRESSING

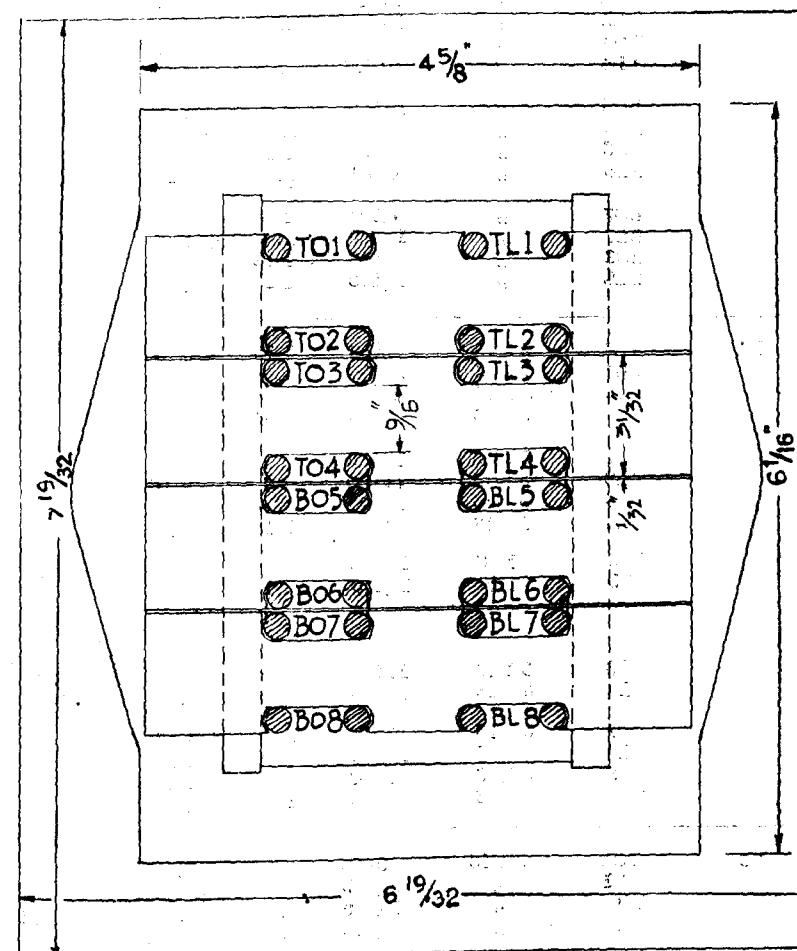
(See para 3.5)



Bottom 8 pairs of cable B_4
 Bottom 8 pairs of cable B_1
 Top 8 pairs of cable T_7
 Top 8 pairs of cable T_6
 Top 8 pairs of cable B_4
 Top 8 pairs of cable B_1
 Bottom 8 pairs of cable T_7
 Bottom 8 pairs of cable T_6
 Top 8 pairs of cable T_5
 Bottom 8 pairs of cable T_5
 Bottom 8 pairs of cable B_2
 Bottom 8 pairs of cable B_3
 Top 8 pairs of cable B_3 *
 Top 8 pairs of cable B_2

* One pair slipped and was re-tensioned at the end.

Top 8 pairs of cable T_5
 Bottom 8 pairs of cable T_5



POSITION OF PAIRS WITHIN A CABLE

Cable No.	Pair No.	Extension (ins.)	Load (lb.)	Slip (in.)	Plus	Minus
B1.	TO1	5 1/16	10,750	1/8	...	- 1/16
	TL1	5 1/8	10,000	3/16	...	- 1/16
	TO2	5 1/16	10,250	1/16	...	- 1/16
	TL2	"	10,000	1/8	...	- 1/16

Cable No.	Pair No.	Extension (ins.)	Load (lb.)	Slip (in.)	Plus	Minus
	TO3	"	10,125	...	+ 1/16	
	TL3	"	10,000	...	+ 1/16	
	TO4	"	10,000	...	+ 1/16	
	TL4	"	10,000	...	+ 1/16	
	BO5	"	10,500	1/8	...	- 1/16
	BL5	"	10,250	...	+ 1/16	
	BO6	"	10,250	1/16	...	
	BL6	"	10,000	...	+ 1/16	
	BO7	"	10,500	1/16	...	
	BL7	"	10,000	...	+ 1/16	
	BO8	"	10,250	1/16	...	
	BL8	"	10,250	1/16	...	
B2.	TO1	5 9/16	10,750	1/16	+ 5/16	
	TL1	5 5/16	11,000	...	+ 5/16	
	TO2	5 5/16	11,000	1/16	+ 4/16	
	TL2	5 9/16	10,750	1/16	+ 8/16	
	TO3	5 5/16	10,750	...	+ 5/16	
	TL3	5 9/16	11,000	1/4	+ 5/16	
	TO4	5 1/16	10,250	1/16	...	
	TL4	5 1/16	10,000	1/16	...	
	BO5	5 1/16	10,000	1/4	...	- 3/16
	BL5	5 1/16	10,250	5-1/16	...	- 5"
						Wires lost sandwich; plate damaged.
	BO6	5 1/16	10,250	1/4	...	- 3/16
	BL6	5 1/16	10,250	1/4	...	- 3/16
	BO7	5 1/16	10,000	...	+ 1/16	
	BL7	5 1/16	10,250	1/16	...	
	BO8	5 1/16	10,250	...	+ 1/16	
	BL8	5 1/16	10,250	1/16	...	
B3.	TO1	5 1/16	10,000	...	+ 1/16	
	TL1	"	10,875	...	+ 1/16	
	TO2	"	10,500	1/8	...	- 1/16
	TL2	"	10,250	...	+ 1/16	
	TO3	"	10,000	1/8	...	- 1/16
	TL3	"	10,125	...	+ 1/16	
	TO4	"	10,000	...	+ 1/16	
	TL4	"	10,000	1/8	...	- 1/16*
	BO6	"	10,000	...	+ 1/16	
	BL6	"	10,500	...	+ 1/16	
	BO6	"	10,250	...	+ 1/16	
	BL6	"	10,000	...	+ 1/16	

* Pair TL4 slipped but was retensioned satisfactorily.

Cable No.	Pair No.	Extension (ins.)	Load (lb.)	(Slip in.)	Plus	Minus
	BO7	"	10,500	1/16	...	
	BL7	"	10,250	...	+ 1/16	
	BO8	"	10,250	...	+ 1/16	
	BL8	"	10,500	...	+ 1/16	
B4.	TO1	5 1/16	10,500	1/8	...	- 1/16
	TL1	"	10,000	1/16	...	
	TO2	"	10,000	...	+ 1/16	
	TL2	"	10,000	...	+ 1/16	
	TO3	"	10,250	...	+ 1/16	
	TL3	"	10,500	1/16	...	
	TO4	"	10,500	1/8	...	- 1/16
	TL4	"	10,000	...	+ 1/16	
	BO5	"	10,125	...	+ 1/16	
	BL5	"	10,000	...	+ 1/16	
	BO6	"	10,750	...	+ 1/16	
	BL6	"	10,125	...	+ 1/16	
	BO7	"	10,625	1/16	...	
	BL7	"	10,500	1/16	...	
	BO8	"	10,750	1/16	...	
	BL8	"	10,500	1/16	...	
T5.	TL1	5 1/16	10,750	1/8	...	- 1/16
	TO1	"	10,500	1/16	...	
	TL2	"	10,500	...	+ 1/16	
	TO2	"	10,250	...	+ 1/16	
	TL3	"	10,250	...	+ 1/16	
	TO3	"	10,625	...	+ 1/16	
	TL4	"	10,500	...	+ 1/16	
	TO4	"	10,500	...	+ 1/16	
	BL5	"	10,250	...	+ 1/16	
	BO5	"	10,500	...	+ 1/16	
	BL6	"	10,750	...	+ 1/16	
	BO6	"	10,500	1/16	...	
	BL7	"	10,500	1/4	...	- 3/16
	BO7	"	10,500	1/16	...	
	BL8	"	10,625	1/16	...	
	BO8	"	10,500	1/16	...	
T6.	TL1	5 1/16	10,750	1/8	...	- 1/16
	TO1	"	11,000	1/8	...	- 1/16
	TL2	"	10,500	...	+ 1/16	
	TO2	"	10,250	...	+ 1/16	
	TL3	"	10,750	1/16	...	
	TO3	"	10,750	...	+ 1/16	
	TL4	"	10,500	...	+ 1/16	
	TO4	"	10,500	...	+ 1/16	

Cable No.	Pair No.	Extension (ins.)	Load (lb.)	Slip (in.)	Plus	Minus
T6 Contd.	BL5	"	10,500	1/8	...	- 1/16
	BO5	"	10,750	...	+ 1/16	
	BL6	"	10,750	1/16	...	
	BO6	"	10,500	...	+ 1/16	
	BL7	"	10,500	1/4	...	- 3/16
	BO7	"	10,750	3/16	...	- 2/16
	BL8	"	10,625	1/16	...	
	BO8	4 9/16	10,750	1/16	...	
T7.	TL1	5 1/16	10,750	1/8	..	- 1/16
	TO1	"	10,750	3/16	...	- 1/8
	TL2	"	10,500	1/16	...	
	TO2	"	10,250	...	+ 1/16	
	TL3	"	10,500	...	+ 1/16	
	TO3	"	10,500	1/16	...	
	TL4	"	10,500	...	+ 1/16	
	TO4	"	10,250	...	+ 1/16	
	BL5	"	10,500	...	+ 1/16	
	BO5	"	10,250	...	+ 1/16	
	BL6	"	10,500	...	+ 1/16	
	BO6	"	10,750	...	+ 1/16	
	BL7	"	10,500	1/16	...	
	BO7	"	10,500	...	+ 1/16	
	BL8	"	10,250	...	+ 1/16	
	BO8	"	10,500	...	+ 1/16	
T8.	TL1	5 5/16	10,500	1/16	+ 4/16	
	TO1	5 5/16	10,750	1/16	+ 4/16	
	TL2	5 5/16	10,500	1/16	+ 4/16	
	TO2	5 5/16	10,750	...	+ 5/16	
	TL3	5 5/16	10,750	1/8	+ 3/16	
	TO3	5 3/16	10,750	1/16	+ 2/16	
	TL4	5 5/16	10,750	1/8	+ 3/16	
	TO4	5 3/16	10,750	1/16	+ 2/16	
	BL5	5 1/8	10,750	1/8	...	
	BO5	5 1/16	10,750	1/16	...	
	BL6	5 3/16	10,750	1/8	+ 1/16	
	BO6	5 1/8	10,750	1/8	...	
	BL7	5 5/16	10,750	1/16	+ 4/16	
	BO7	5 5/16	10,500	1/16	+ 4/16	
	BL8	5 1/16	10,750	1/16	...	
	BO8	5 1/16	10,500	1/16	...	

Appendix III.

CALIBRATION OF JACKS AND GAUGES.

50-Ton Jack No. R. 26 (Ram Area, 19.6 sq. in.).

Actual load from Testing Machine	Gauge reading	(Prestressing gauge No. 1. on Ram area 5.9. sq. in. used on the 14th and 15th June except on the evening of the 15th)
0.5 tons	No reading	
1.0 "	No reading	
1.5 "	800 lb	
2.0 "	1100 "	
2.5 "	1350 "	
3.0 "	1700 "	
3.5 "	2000 "	
4.0 "	2250 "	
4.5 "	2750 "	
5.0 "	2900 "	
5.5 "	3250 "	
6.0 "	3500 "	
6.5 "	3800 "	
7.0 "	4000 "	
7.5 "	4250 "	
8.0 "	4600 "	
8.5 "	4900 "	
9.0 "	5250 "	
9.5 "	5500 "	(corresponding to the working load on 14th & 15th June)
10.0 "	5750 "	
11.0 "	6500 "	
12.0 "	7000 "	
13.0 "	7800 "	
14.0 "	8250 "	
15.0 "	8950 "	
16.0 "	9500 "	
17.0 "	10125 "	
18.0 "	10750 "	
19.0 "	11375 "	
20.0 "	12000 "	
21.0 "	12625 "	
22.0 "	13250 "	
23.0 "	13750 "	
24.0 "	14375 "	
25.0 "	15000 "	

26.0 tons	15635 lb	
27.0 "	16250 "	
28.0 "	16800 "	
10.0 "	23 tons	(gauge No. 5681546; load on
12.0 "	28 "	ram area 39.5 sq. inches
14.0 "	32 "	used towards the evening
16.0 "	37 "	of 15-6-52 and on 18-6-52)
18.0 "	43 "	
20.0 "	47 "	
22.0 "	51 "	
24.0 "	57 "	
26.0 "	63 "	
28.0 "	70 "	
30.0 "	74 "	(corresponding to total load
		of 155 tons on 15-6-52)
32.0 "	80 "	
34.0 tons	86 "	(corresponding to total load
36.0 "	92 "	of 170 tons on 18-6-52)
38.0 "	97 "	
40.0 "	102 "	
42.0 "	108 "	
44.0 "	114 "	
46.0 "	119 "	
48.0 "	124 "	
50.0 "	130 "	

50 Ton Jack No. R 27 (Ram area 19.6 sq. in.)

Actual load from Testing Machine	Gauge reading	(Prestressing gauge No. 1. Load on area 5.9 sq. in. used on the 14th and 15th June except on the evening of the 15th)
0.5 Tons	No reading	
1.0 "	375 lb	
1.5 "	1000 "	
2.0 "	1250 "	
2.5 "	1500 "	
3.0 "	1750 "	
3.5 "	2125 "	
4.0 "	2425 "	
4.5 "	2750 "	
5.0 "	3000 "	

5.5 Tons	3375 lb	
6.0 "	3625 "	
6.5 "	4000 "	
7.0 "	4250 "	
7.5 "	4500 "	
8.0 "	4750 "	
8.5 "	5125 "	
9.0 "	5500 "	(Corresponding to the work-
9.5 "	5750 "	ing load on 14th and 15th
10.0 "	6000 "	June).
11.0 "	6500 "	
12.0 "	7250 "	
13.0 "	7875 "	
14.0 "	8500 "	
15.0 "	9125 "	
16.0 "	9875 "	
17.0 "	10550 "	
18.0 "	11125 "	
19.0 "	11750 "	
20.0 "	12500 "	
21.0 "	13125 "	
22.0 "	13750 "	
23.0 "	14375 "	
24.0 "	15100 "	
25.0 "	15750 "	
26.0 "	16000 "	
26.0 "	69.5 tons	(Gauge No. 5681546—load on
28.0 "	72.0 "	Ram area 39.2 sq. in. used
		towards the evening of
		15-6-52 and on 18-6-52).
30.0 "	80.0 "	(Corresponding to the total
32.0 "	85.0 "	load of 155 tons on 15-6-52).
		(Corresponding to the total
		load of 170 tons on 18-6-52).
34.0 "	92.0 "	
36.0 "	97.0 "	
38.0 "	102.0 "	
40.0 "	108.0 "	
42.0 "	114.0 "	
44.0 "	120.0 "	
46.0 "	124.0 "	
48.0 "	130.0 "	
50.0 "	136.0 "	

100-Ton Jack No. 1 (Ram area 41.26 sq. in.)

Actual load from Testing Machine	Gauge reading	(Prestressing gauge No. 1 on Ram area 5.9 sq. in. used on 15-6-52 except in the evening)
0.5 Tons	No reading	
1.0 "	"	
1.5 "	500 lb	
2.0 "	600 "	
2.5 "	750 "	
3.0 "	950 "	
3.5 "	1125 "	
4.0 "	1250 "	
4.5 "	1450 "	
5.0 "	1600 "	
5.5 "	1750 "	
6.0 "	1875 "	
6.5 "	2000 "	
7.0 "	2200 "	
7.5 "	2250 "	
8.0 "	2500 "	
8.5 "	2650 "	
9.0 "	2750 "	
9.5 "	2850 "	(Corresponding to the working load on 15-6-52).
10.0 "	3000 "	
11.0 "	3200 "	
12.0 "	3350 "	
13.0 "	3600 "	
14.0 "	3750 "	
15.0 "	4125 "	
16.0 "	4450 "	
17.0 "	4700 "	
18.0 "	5000 "	
19.0 "	5250 "	
20.0 Tons	5500 "	
21.0 "	5850 "	
22.0 "	6150 "	
23.0 "	6400 "	
24.0 "	6750 "	
25.0 "	7050 "	

DESTRUCTION TEST OF PRESTRESSED GIRDER

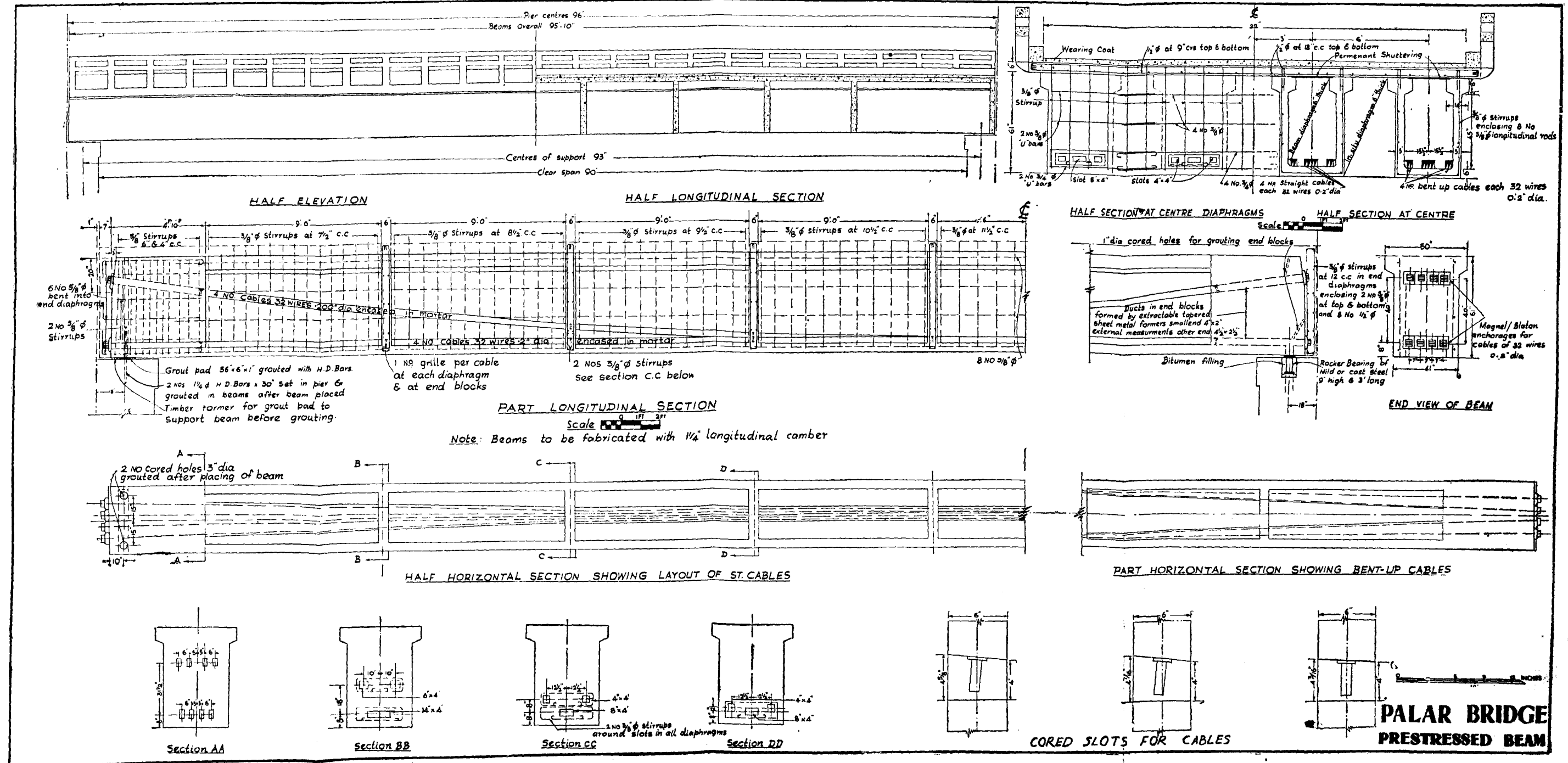
26.0 Tons	7450 lb	
27.0 "	7600 "	
28.0 "	8000 "	
29.0 "	8250 "	
30.0 "	3500 "	
10.0 "	23 Tons	(Gauge No. 5757009 on Ram area 82.52 sq. in. used towards the evening of 15-6-52 and on 18-6-52).
12.0 "	25 "	
14.0 "	30 "	
16.0 "	32 "	
18.0 "	36 "	
20.0 "	39 "	
22.0 "	44 "	
24.0 "	47.5 "	
26.0 "	51 "	
28.0 "	55 "	
30.0 "	58 "	
32.0 "	62 "	
34.0 "	65 "	
36.0 "	68 "	
38.0 "	72 "	
40.0 "	75 "	
42.0 "	80 "	(Corresponding to total load of 155 tons on 15-6-52).
44.0 "	82 "	

100-Ton Jack No. 2 (Ram area 41.26 sq. in.)

Actual load from Testing Machine	Gauge reading	(Prestressing gauge No. 1 Ram area 5.9 sq. in. used on 15-6-52 except in the evening)
0.5 Tons	No reading	
1.0 "	250 lb	
1.5 "	375 "	
2.0 "	500 "	
2.5 "	625 "	
3.0 "	750 "	
3.5 "	900 "	
4.0 "	1050 "	
4.5 "	1250 "	
5.0 "	1375 "	
5.5 "	1500 "	
6.0 "	1625 "	
6.5 "	1750 "	
7.0 "	1875 "	
7.5 "	2000 "	

NAMBIAR & RAJ ON DESTRUCTION TEST OF
PRESTRESSED GIRDER

8.0	"	2125	lb	
8.5	"	2280	"	
9.0	"	2500	"	
9.5	"	2600	"	
10.0	"	2750	"	(Corresponding to the working load on 15-6-52).
11.0	"	3100	"	
12.0	"	3500	"	
13.0	"	3750	"	
14.0	"	4000	"	
15.0	"	4375	"	
16.0	"	4700	"	
17.0	"	5000	"	
18.0	"	5250	"	
19.0	"	5625	"	
20.0	"	5950	"	
21.0	"	6250	"	
22.0	"	6500	"	
23.0	"	6750	"	
24.0	"	7125	"	
25.0	"	7450	"	
26.0	"	7750	"	
10.0	"	21	tons	(Gauge No. 5757009 on Ram area 82.52 sq. in. used towards the evening of 15-6-52 and on 18-6-52).
12.0	"	25	"	
14.0	"	29	"	
16.0	"	32.5	"	
18.0	"	35	"	
20.0	"	39	"	
22.0	"	42	"	
24.0	"	45	"	
26.0	"	48.5	"	
28.0	"	52.5	"	
30.0	"	56	"	
32.0	"	59	"	
34.0	"	71	"	
36.0	"	67	"	
38.0	"	71	"	
40.0	"	75	"	(Corresponding to total load of 155 tons on 15-6-52).
42.0	"	79	"	
44.0	"	83	"	(Corresponding to total load of 170 tons on 18-6-52).
46.0	"	88	"	



BRIDGING INDIA'S RIVERS

**AN ACCOUNT OF
SOME OF THE BRIDGES
BUILT UNDER THE NATIONAL HIGHWAY SCHEME**

FOREWORD

In December 1943, the Chief Engineers of the various provinces and states of India met at Nagpur to consider ways and means of planning future road development after the last war. The plan drawn up by them has become famous as the Nagpur Plan and forms the basis of road development under the Five Year Plan too.

To implement the Nagpur Plan, the Central Government accepted responsibility, with effect from the 1st April 1947, for the construction and maintenance of a provisional system of National Highways defined as "main highways running through the length and breadth of India, connecting ports, foreign highways, capitals of Provinces and large States, and including roads for strategic movements for the defence of India".

Although the alignment of these highways is still provisional, new construction and improvement works on them have been going on apace. One of the most important class of works done on these highways has been the construction of bridges over big rivers hitherto left unbridged on account of the large financial commitments involved. Many such bridges have been built and many more are under construction.

For the information of the interested public and the profession a series of articles on the bridges completed (or nearing completion) on National Highways since 1947, under the National Highway Scheme, was published in the Transport-Communications Monthly Review during 1952. These articles are now published together in one place for easy reference. To make the publication useful, the I. R. C. loading on which the design of these bridges has been based is published for the first time. (Plate I)

A list of the bridges described in the articles with their cost is appended. The list is not comprehensive as many other bridges are still in progress or have recently been completed. Also the articles do not describe many of the smaller bridges constructed

during the last five years although these presented many interesting professional problems.

The designs adopted for many of the bridges described are new to India and the foundation problems met with in some cases were as difficult as any that have been tackled in the past. Detailed descriptions of some of these problems and how they were tackled have been published as Papers in the Journal and may be referred to by those interested after reading this review.

List of Bridges Described.

(See Plate II)

Name.	Length feet.	Cost (lakhs of Rupees)
1. Barakar Bridge.	607	29.6
2. Pennar Bridge.	2,426	25.0 (Estimate)
3. Kathjuri Bridge.	2,828	45.9
4. Cauvery Bridge.	2,634	25.9
5. Kuakhai Bridge.	1,696	23.2
6. Tansa Bridge.	345	4.3
7. Champavati Bridge.	1,033	13.2 (Estimate)
8. Taldanga Bridge.	317	3.6
9. Kaljani Bridge.	455	9.1
10. Raidak Bridge.	455	9.1
11. Gadadhar Bridge.	200	4.0
12. Poonpoon Bridge.	315	9.7
13. Bakhra Bridge.	227	3.8
14. Kitchinia Bridge.	416	5.7
15. South Pennar Bridge.	568	4.0

BRIDGING INDIA'S RIVERS

Hints on Preparing

RECONNAISSANCE REPORTS ON BRIDGE SITES

During every road-engineer's professional career it will probably fall to his lot on many occasions to carry out reconnaissance surveys for bridge sites. The bigger and more important the bridge, the higher will be the responsibility devolving upon the reconnoitering officer, for on his reports will mainly depend the final selection of the site. It is, therefore, important that the engineer should have a good grasp of the fundamental factors which help him arrive at correct decisions. It is the purpose of this article to provide the engineer with an **aide-memoire** on what to look for and what to include in his report so as to make it useful.

What is a Good Site?

2. The characteristics of an ideal site for a bridge are : -
 - (1) A straight reach of the river ;
 - (2) Steady regimen of the river and absence of serious whirls or cross currents ;
 - (3) A narrow and well defined channel ;
 - (4) Suitable high banks above high flood level on each side ;
 - (5) Rock or other compact and fairly unerodable foundations close to the bed level ;

- (6) Secure and economical approaches which should not be very high, long, or liable to flank attacks of the river and its spills during floods; nor should the approaches involve obstacles, e.g., hills, frequent drainage crossings, sacred places, graveyards, congested or built-up areas needing viaducts, or troublesome land acquisition;
- (7) Reasonable proximity to a direct alignment of the road to be served, i.e., avoidance of long detours;
- (8) Absence of sharp curves in the approaches;
- (9) If it is unavoidably necessary for the approaches of a bridge to cross the spill-zone of a river, they should, while proceeding through the spill zone towards the river, face downstream and not upstream. Facing upstreams will cause heading up, pocket formation, and danger to the approaches;
- (10) Absence of costly training works and, where such works are unavoidable, the possibility of executing them largely in the dry;
- (11) Avoidance of excessive construction work under water.

It is hardly necessary to mention that the "ideal" site never exists in reality. At every site, one or more of the ideal conditions is lacking and the object of a reconnaissance ("recce" for short) is to select the least objectional site.

3. In making this selection, the relative importance to be attached to any one factor may vary considerably according to the nature of the bridging problem. The best compromise is a matter of judgement but that judgement can only be applied after careful study of relevant data. It is the business of the "recce" officer to collect that data and to **record it**.

"Rejection" Precedes Selection

4. The officer deputed to reconnoitre for a bridge site should first study at home the largest available map of the area. He should tentatively fix the limits of the terrain within which his examination is to be conducted and make a tentative selection, on the map, of the sites and locations that seem to warrant inspection. The best method of doing this is by a process of

"rejection", i.e., by considering the topography of the reach of the river as a whole and rejecting such stretches where the chances of finding a suitable site for a bridge are negligible.

5. After eliminating unsuitable sites the "Recce" Officer should concentrate upon the remainder, giving prior attention to the 'probable' site nearest to the line along which, other things being favourable, the proposed road would run.

6. Having decided the site or sites to which reconnaissance activities should be directed, the Recce Officer should calculate the catchment areas at the probable sites from maps in his possession and get together any other information readily available with him about flood levels, discharges, waterways, etc., relating to the sites and to any bridges on the same river in the vicinity.

7. He should also take steps to collect meteorological information like annual rainfall and seasonal distribution, storm data, direction of prevailing and strongest winds, range of temperature (seasonal and daily), and humidity conditions.

If there are existing road or railway bridges not very far upstream or downstream of the site he is to examine, he should write and ask for particulars of these bridges.

Field work

8. Now the Recce Officer is ready to start his field work. His party should include, if possible, the officer who is to be responsible for carrying out the subsequent detailed investigation and preparation of the project.

9. **Equipment:**—The equipment which the party will need during field work includes—

- (1) Tapes and survey chains with plenty of arrows;
- (2) Half a dozen bamboo poles, each shod with steel, marked in black and white for feet, and carrying flags at the top;
- (3) A clinometer or Abney level;
- (4) A dumpy level with staff or, alternatively, a straight edge, and an ordinary mason's level;
- (5) A prismatic compass or sextant;

- (6) A stop watch or, in its absence, an ordinary watch with a second-hand;
- (7) A crowbar and a one-inch diameter steel rod, about 12 ft long, with one end pointed, and a sledge hammer;
- (8) Pegs and mallets.

10. **Data to be Collected.** The field examination of the site should be rapid but sufficiently thorough to enable the officer to include in his reconnaissance report all the information indicated in the following paragraphs—or at least, most of this information.

11. **Preliminaries.** The officer should first write down the following:

- (1) Name of the river he is to examine, of the highway requiring the bridge, and the mileage of the probable bridge site from the nearest town or identifiable village.
- (2) The name of the State and the District;
- (3) Map reference of the crossing point or the approximate latitude and longitude scaleable from maps;
- (4) Location of the nearest G. T. S. Bench Marks with their levels;
- (5) Nature of the river (perennial, tidal, snow-fed, or rain-fed, etc.). In the case of perennial rivers, indicate low water level and its greatest depth.
- (6) Description (with dimensioned sketch showing foundations) of any existing bridge or causeway over the same river in the vicinity, giving reason if the work is unsatisfactory.
 - (a) For an existing causeway, indicate the maximum depth of submersion, the height of the lowest point on the causeway above the lowest bed level, ventage if any, frequency and duration of interruptions to traffic.
 - (b) For an existing bridge, give the known clearance above the low water level, lowest bed level, and H. F. L. If any trouble has been caused by scour or flank attacks on approaches, give particulars.
 - (c) When a bridge is to replace a causeway and is sited close to it, other things being equal, a site upstream of the causeway is generally to be preferred.

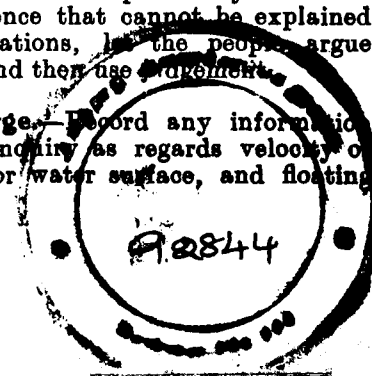
† + 201

- (d) If the proposed bridge is to be near another existing bridge on the same river, the piers of one should not cause difficulties for the piers of the other by giving rise to cross currents or scour.

He should accompany this preliminary note with a sketch showing on the sketch itself the information listed above. He should also show the north point and the direction of flow of all streams. (See para. 21 below).

12. **Hydraulic Data.** The following hydraulic data should be collected:—

- (1) **Catchment**—(a) A traced index map showing the catchment area (suitable scale 1 in. to a mile or smaller) indicating, if possible, rough boundaries of hilly country, plains, forest, cultivated or barren land, etc. and the general nature of the soil in the area.
 - (b) The worked-out catchment area in square miles from 1 in. or $\frac{1}{2}$ in. G. T. S. maps at (roughly) the site envisaged for the new crossing.
- (2) **H. F. L.**—(a) High floods generally leave water-marks on tree trunks and these are visible for a number of years after the floods. The marks consist of rings of silt (with or without twigs sticking to them) around the trunks of trees. Carefully examine old trees and look for these marks.
 - (b) Also ascertain by local enquiry (by individual, and **not** collective, interrogation of old inhabitants) the points on the ground (or on trees or structures) scattered over a wide area, that has been reached by the highest flood level in living memory. Show these points on sketch plans and cross-sections, not necessarily to scale. Then link the levels of all these points to a common datum. Where there is a considerable difference in the levels reported by the local people, a difference that cannot be explained by afflux calculations, let the people argue the matter out and then use judgement.
- (3) **Velocities and Discharge.** Record any information available from local enquiry as regards velocity of flow, slopes of bed or water surface, and floating debris.



- (a) If there is a nearby existing causeway, the extent of the afflux caused by it should be ascertained, if possible. Rough discharge figures can often be estimated from this data.
- (b) If the reconnaissance is done while the river is carrying an appreciable discharge, the time taken by a log thrown into the centre of the stream to float down a measured distance gives a rough indication of the velocity at that level which should be stated.
- (c) If constants of any of the catchment and discharge formulae are known for the area, give figures for discharge based on these.

13. **Siting.** Prepare a sketch map, not necessarily to scale, showing the position of the sites examined and assess the advantages and disadvantages of each site and give reasoned preferences for the sites in the order of merit.

Detailed Examination

14. Sites not definitely rejected must all be examined and assessed for their merits. If any one site is unmistakably superior to the others, detailed examination may be confined to that site only.

Classify information as below :—

- (1) Give a sketch, preferably to scale, of the cross-section of the river at the site considered. A dumpy level can be used for this but on "reco" work it is preferable to use an Abney level or a clinometer or even a ghat-tracer. Even without these instruments, much useful information can be obtained by eye observation alone with the help of marked bamboo poles, a tape, and a mason's level. Cross-sections can be roughly gauged from temporary siting-flags erected on each bank.
- (2) Mention whether the reach is straight or curved; give the length of the "straight" and the approximate position of the site on the "straight".
- (3) Show in the sketch all tributary streams entering the river within a reasonable distance (say 5 to 10 times the width of the river) upstream of the site being examined. Give the approximate discharge of these tributaries as ascertained from the catchment area map.
- (4) Ascertain, if possible, the depth of the deepest scour hole below H. F. L. and mention whether the hole is in the normal channel, against a hard bank obstruction, at a curve, or "local," caused by a whirlpool.
- (5) In wide, meandering alluvial rivers, indicate the width between stable banks, the position and widths of "nodes", i.e., the points at

- which the stable banks come closest to each other, and the width and depth of the channel during "dry" weather, ordinary floods, and high floods. Also indicate roughly the radii of the larger meanders taken from the map.
- (6) Show in the sketch plan the conditions of the approaches for 1000 ft on each side of the site being examined and sketch in the alignment of the proposed approach road. Run a line of levels with an Abney level, clinometer, dumpy level, or by boning rods to eye measurement, along this alignment and use a compass to show roughly the tangents of curves.

15. **Bed Material and Foundation :—**Indicate on the cross-section the bed material on the surface, and also a little below it as ascertained by probing with an iron rod, and say whether the bed is fairly homogeneous.

- (1) If rock is exposed in the bed, indicate its position.
- (2) In the case of sand, record whether it is quartzzy, very micaceous, very coarse, coarse-medium, fine, or very fine.
- (3) In the case of clay, record whether it is sticky, stiff, loamy, loose, or soft.
- (4) In the case of boulders, record whether they are large or small, angular or rounded, and give their approximate size.

Note—State also the nature of the soil surface in the approaches.

- (5) Then, probe deeper into the (dry) bed of the river with a steel bar (1 in. diameter is usually enough), using a sledge hammer and note the movement per blow, or the number of blows per foot sinkage. If the bed material alters within the range of the bar (12 ft length is convenient), the fact can usually be recognized and sometimes samples remain attached to the point of the bar. Aim at finding rock.

16. **Banks :—**State whether the material of the banks is rock, clay, loam, very sandy soil, or pure sand, etc.

If any rock projects from a bank into the river in such a manner as to deflect the current or cause serious whirls, give its position. If any bank is seriously eroded, or silted, indicate the position.

17. Supplies of Materials :—State what building materials can be made available, their quantity if limited, quality, source of supply, distance and means of communication, and methods of transport.

Specially mention the following :—

- (1) earth for embankments ;
- (2) puddle clay ;
- (3) building stone, hard filling, pitching stone ;
- (4) stone suitable for concrete aggregate ;
- (5) bricks, brick-clay for local manufacture ;
- (6) lime ; especially mention kankar lime, if available, and its quality ;
- (7) sand ;
- (8) timber.

18. Labour, Staff, Amenities. Classify the required information as follows :—

- (1) Skilled or unskilled labour ; available locally or to be imported and if to be imported, from where ;
- (2) Existing facilities for housing labour and staff at or close to the site of work ; need for building such accommodation. Mention building facilities, especially what kind of roofing can be provided.
- (3) Nearest village or market from which supplies can be obtained ;
- (4) Healthiness of the site ; quality and quantity of drinking water supply ;
- (5) Any special services obtainable, e.g., medical facilities, piped water supply, post and telegraph office, and electricity for domestic or power use (state A. C., or D. C., voltage, phases, frequency).

19. Writing the Report. The report should be brief and to the point but must not omit anything relevant. Clear, brief, and

crisp reports, accompanied by sketch maps, give a far better picture than do long narratives :

- (1) The Recce Officer will have to apply a broad vision and commonsense to the problem taken as a whole.
- (2) If only one site is predominantly superior to all others, the Recce Officer should recommend that any subsequent investigations be confined to that site only.
- (3) If several sites merit consideration, he should give his own order of their respective merits.
- (4) In either case he should recommend the scope of the subsequent detailed investigations and surveys needed to obtain the full data required for the design of the bridge.
- (5) The scope of the detailed investigation may vary from a very brief local survey for determining locations, reduced levels, and sections in the simpler cases, to fairly comprehensive investigation of topographical, hydrological, and geological data including exploratory borings of the subsoil, in the more complicated cases.
- (6) For economy as well as speed it is necessary that the scope should be limited to actual requirements and intelligently drawn up reconnaissance reports are a great help in this.
- (7) **Sketches.** The most important part of the Report should be the sketches. These should give not only information about the site under consideration but of the
 - (i) configuration of the river and its tributaries, for a distance of from five to ten times its width, upstream and downstream of the site ;
 - (ii) rough levels upstream and downstream of the whole length shown in the sketch ;
 - (iii) alignment and levels of the approaches to each site ;
 - (iv) position of nallahs, hills, tanks, lakes, temples, and all obstructions that would lie in the way of locating alternative approach roads to the site.

(8) **Points the Recce officer should remember.**

- (i) He is concerned with selecting the best possible bridge site but this does **NOT** always mean the place where the cheapest bridge can be built. The best site is generally the one where the cheapest bridge **with its approaches** can be built.
- (ii) He must think of the road and avoid long diversions if possible.
- (iii) He must also think of the road needs of the traffic that is going to use the road for the next thousand years and avoid sharp curves, steep gradients, blind approaches etc.
- (iv) He must also think of the origin and destination of traffic that will cross the river after the bridge is built and locate his site as far as possible to suit the needs of present, and more important, future needs of traffic.
- (v) But he must not be influenced in his decision by local vested interests either in land, or in traffic.
- (vi) In brief, the recce officer is to select the best site, not only the best site technically, but the best site having in view the long-term prosperity of the country. The recce officer has a difficult and important task to perform.

20. **Mistakes.** It may help the recce officer in his work and in writing his report to think of mistakes that have been made in the past by some of his predecessors. A few of the worst mistakes are listed below :—

- (1) Bridge sited immediately downstream of a junction of two big rivers. [This mistake comes of looking only at a site and not upstream.]
- (2) Bridge undermined after construction owing to regression of soft rock in the river bed. [This mistake comes through the recce and survey officers not looking immediately downstream where there were a series of waterfalls rapidly travelling up the soft-rock bed of a river.]

- (3) The designs of two bridges were interchanged and each was built over the stream for which the other was intended. [This extraordinary mistake occurred because the recce and survey officers did not take enough trouble to show the exact sites of the bridges on their plans with reference to neighbouring villages, the streams themselves, and the approaches. The construction officer, of course, also acted without intelligence.]
- (4) Site involving deep foundations selected when there was a site with rock at no great depth only 700 ft downstream. [This mistake would not have occurred had the recce officer sketched the course of the river downstream because there were surface indications (configuration of the banks and rocks on the banks on each side) that indicated the probable presence of rock at no great depth below the bed.]
- (5) Bridge much longer than necessary was built. [This mistake occurred because the recce officer did not notice that the wide spread of the river from bank to bank at the selected site was due to the presence of an old causeway which had frequently been outflanked and extended in the past. Half a mile upstream and half a mile downstream the river had accepted a much narrower bed.]
- (6) Afflux caused by guide banks flooded some villages. [This mistake occurred because the recce, survey, and construction officers did not calculate probable affluxes and did not examine the levels of the country in the immediate neighbourhood of the site.]
- (7) More money was spent on the approaches to a bridge than on the bridge itself. Had the bridge been built 200 ft upstream, the total cost would have been halved.

The mistakes quoted are mistakes that have actually occurred in the past. The list could be made much longer. In nearly every case the mistake arose because, first the recce officer and then the survey and construction officers, had their noses glued to a particular site and neither looked upstream nor downstream, nor did they look to the right or the left hand approaches to the bridge. They were obsessed with a particular **BRIDGE SITE**,

21. This article is headed "Reconnaissance Reports on Bridge Sites". Let the recce officer observe the plural in the word "sites" and look upstream and downstream of the river and to his right and left on the banks. Let him then **make sketches of what he sees**, putting in names of villages, a north point, direction of flow of his river, sketches of hills and nallahs, temples and graveyards, in fact of obstructions of all kinds. Let him

- (1) Use his eyes, and
- (2) Record what his eyes have seen.

1. THE BARAKAR BRIDGE

Nearly exactly two hundred and fifty miles from Calcutta, the Grand Trunk Road (National Highway No. 2) crosses the Barakar river in Bihar over a brand new bridge which came into use only last year. Travelling over it the motorist will be tempted to stop and get to have a close look at this imposing concrete structure. Outwardly there will not be much to see of the intricate pattern of mighty steel rods that have been woven into the hard concrete to form the graceful superstructure that is so pleasing to the eye. He may, however, possibly feel that the approaches are on very high embankments and that the piers are high and lanky and rather thinner than usual.

If he enquires he will learn the reason why—a few years hence, he will be told, the road and the bridge will be in the middle of a lake which will flood the surrounding country when a dam under construction a few miles away is completed. Then the piers will show only a few feet above the lake waters. When this happens it will be worth the motorist's while to go again and stand on the bridge in the middle of the lake.

The new bridge lies between the ruins of two old bridges. A few hundred feet downstream stand the remains of the abutments of a masonry bridge—9 arches of 50 ft—which collapsed at the age of 71 years in 1911 after a week of incessant rain which brought an "unprecedented" flood (H.F.L. 1212.7 ft).



Remains of a collapsed abutment.

A screw-pile bridge (5 spans of 105 ft) was built in 1914 just before the first World War. History repeated itself in September 1946 when a wet week culminated in 10 inches of rain in one day and another "unprecedented" flood (H.F.L. 1216.3 ft this time), accompanied by a cyclonic wind. Three out of the five spans disappeared overnight. Two spans are still there.

THE BARAKAR BRIDGE



[Photo : B. B. & Jessop Const. Co., Ltd.]

A view of the completed bridge. In the background the remaining two spans of the screw-pile bridge are still seen.

The new bridge is of the cantilever-cum-suspended span type in reinforced concrete designed to the specifications of the Indian Roads Congress for 2 lanes of Class A loading and, alternatively, one lane of Class AA loading. This particular type of span was adopted as it does away with the possibility of unknown stresses occurring on account of slight movements of the supports. Moreover, materials required for this type of construction were easily available for this bridge within economical distances.

The main dimensions of the bridge are :

Overall length face to face of abutments, 607 ft; distance apart of centres of piers, 92 ft; height of pier shafts from plinth level to bearing seat, 44 ft; width of pier shaft, 5 ft; length of pier shaft, 28 ft 4 in.; width of roadway, 24 ft; maximum depth of main girders, 10 ft at piers; minimum depth of main girders and suspended span at the centre of the span, 4 ft; present maximum high flood level, 1219.56; new high water level after the Tilaya Dam is built, 1230.00 (this accounts for the extra height—10 ft 6 in.—of the piers); catchment area above the bridge site, 276 sq. miles.

Costs

The bridge has cost Rs 1,62,000 for foundations, Rs 4,88,700 for substructure, and Rs 4,59,500 for superstructure. The approaches (embankment and shore spans) have cost Rs 17,86,000. The earthwork was done entirely by manual labour. As a measure of economy eleven shore spans (six arches of 46 ft span on the Calcutta side and five on the Delhi side) were introduced on either side of the main bridge where the lake waters will be deep.

The total cost of the bridge was Rs 29,62,205 which works out to Rs 1,753 per running foot, leaving out the cost of the approaches, or Rs 73 per sq. ft. of the roadway.



A portion of the Screw-pile bridge

Cellular Construction.—With the exception of the suspended span, the entire bridge, i.e., piers, abutments, and superstructure are built in cellular construction.



Calcutta side span under construction.

The pier shaft which looks solid is, in reality, a 3-cell vertical cantilever beam with high-grade concrete walls only 8 in thick, reinforced with vertical bars, one inch square, at 14½ in. centres. For stability the hollow cells are plugged with plum concrete to various heights to suit design requirements. Because of the great height exposed to the water current, the piers have been made very slim so as to offer the minimum obstruction to the flood waters.

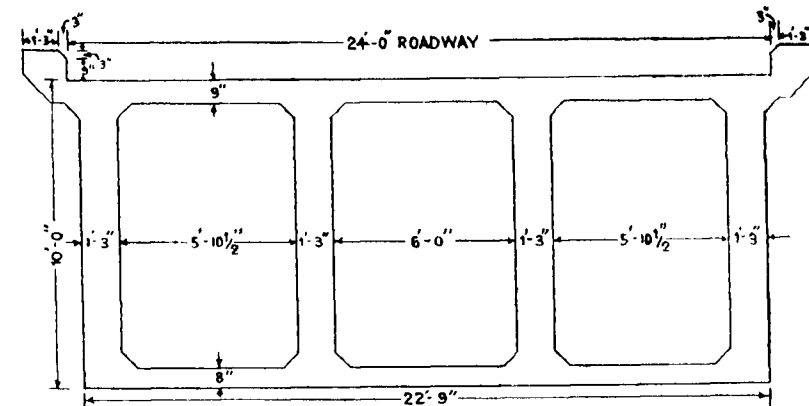
The abutment walls vary from 8 in. to 17 in. in thickness. The abutment cells were plugged solid with sand.

Very careful calculations were made in the case of the piers and abutments for "silo" stresses which might be caused by the bursting action of the wet concrete or granular sand plugs.

One abutment and two piers were constructed with open foundations, the other foundations being designed for cofferdam construction. Taking advantage of the lessons of the two previous bridge failures, the designers took all foundations down to solid, hard rock. Up to plinth level the foundations are in reinforced mass concrete.

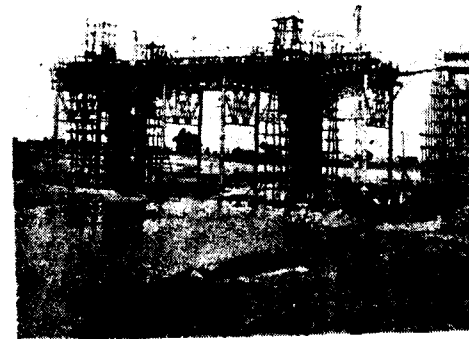
Superstructure—The main spans are fully cellular, the top slab being 9 in. thick (with 3 in. extra wearing coat), the bottom slab 5 in. to 8 in. thick and the four webs 9 in. to 15 in. thick with diaphragms so arranged that the whole span acts as one large beam. The gross steel area near the centre of the span is 135 square inches and the design provides for the shear to be taken either by the concrete alone or by the web reinforcement alone.

The expansion bearings of the main spans consist of a row of four Mehanite high duty cast iron bearings of a design specially developed for this bridge.



CROSS SECTION OF THE MAIN SPAN
— AT PIER SUPPORT. —

The suspended span (25 ft) is non-cellular, designed as a standard, straight forward beam, the only special feature being its closely machined, pressure lubricated, phosphor-bronze-plate expansion bearings.



A view of the main centring.

The railing is pre-cast with a few main uprights monolithic with the slab.

building of the shafts would have been one continuous pour from the plinth to the bearings.

Lifting Shuttering.—The pier shafts were constructed by means of specially designed lifting shuttering operated inside guide frames. If rapid-hardening cement had become available the

The shuttering for the main span was supported on specially designed steel framing and centring adjustable for the varying levels of the river bank.

2. THE PENNAR BRIDGE

Recently a Frenchman who had lived in Calcutta for about five years asked us when it would be possible for motorists to drive down from Calcutta to Madras. The questioner had in mind the numerous unbridged rivers between these two cities. We have counted on a map these obstructions to motor traffic and find that the two new bridges over the Kathjuri and the Kuakhai on this highway hardly touch the fringe of the problem. There are still another eleven major rivers left unbridged. The Pennar is one of these and work on a bridge across this river is now in progress.

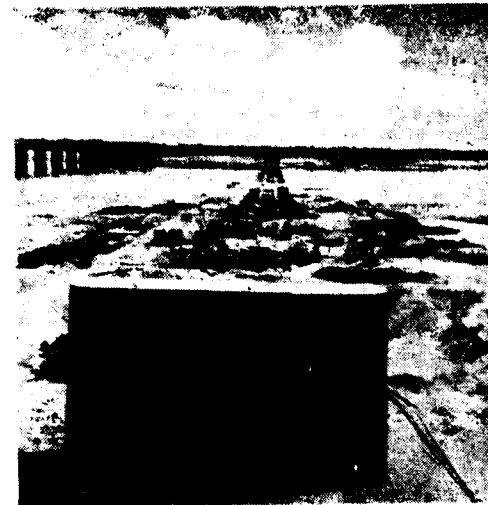
The need for all these bridges is self-evident; the only question is of priorities. Many considerations go to decide this question, the main being the volume of traffic on any particular section of the highway, and the cost of providing a river crossing.

In this particular case the traffic across the river is catered for by a weir-cum-causeway over the Pennar. Traffic here is heavy as this causeway is the only link between the two heavily populated parts of the Nellore district. The river divides this fertile deltaic district into two almost equal halves and the main market towns are on opposite sides of the river.

A Death Trap

Traffic over the weir is, however, unsafe as there is always a likelihood of some flow over it. Interruptions to traffic are frequent and traffic may be held up on 30 to 90 days in a year. The presence of the weir-causeway therefore became worse than no river crossing at all, for it had lured many a traveller to his death. Under these circumstances the Madras Government gave a very high priority to this bridge.

The site and the span length were almost fixed by circumstances for the designers who were not left with much discretion. As would be expected, for many miles upstream of the weir, the river is very wide. Three furlongs below the weir there is a railway bridge and the irrigation engineers did not agree to another bridge being put up between these two works. The only course open to the highway engineers was to build downstream of the railway bridge and as near to it as possible so as to take advantage of the existing river training works. Further, the railway engineers insisted that the road bridge piers should be so sited that no cross-currents were set up between the two bridges. Consequently, each pier of the highway bridge was



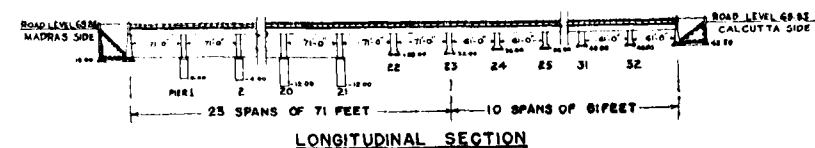
View from the Madras side Abutment
Railway Bridge is seen on the left.

located exactly in line with the corresponding pier of the existing railway bridge. Both bridges have the same number of spans and the same length between abutments. The highway bridge is sited 300 ft below the railway bridge.

As most of the piers are founded on clay, reinforced concrete T-beam design with simply supported free ends was adopted for the superstructure. Commencing from the south side the bridge has 23 spans of 71 ft and 10 spans of 61 ft each (clear). The first 21 piers have well foundations, but in the two abutments and the last 11 piers open foundations were adopted as laterite rock was found at a depth of 3 to 5 ft below the bed in most of these cases. The bridge is designed for two lanes of I.R.C. Class A loading.

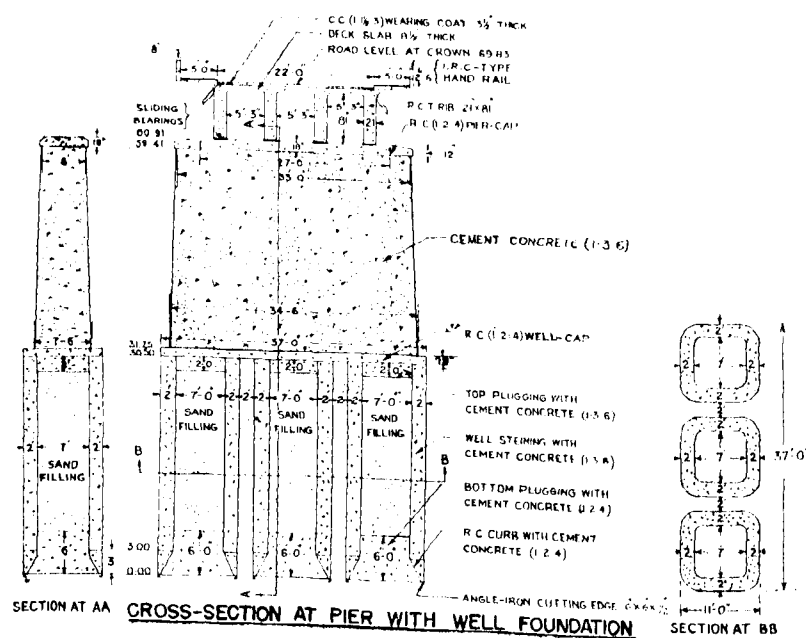
The main dimensions of the bridge are :

Overall length face to face of abutments, 2,426 ft ; distance apart of centres of piers : 23 spans, 77 ft and 10 spans, 66 ft ;



width of pier shaft, 6 ft in the longer spans and 5 ft in the shorter spans ; length of pier shaft in plan 33 ft for longer spans and 32 ft for shorter spans ; width of carriage-way, 22 ft with two 5-ft footpaths ; T-beam web 81 in. by 21 in. (62 in. by 20 in. in the shorter spans), deck slab thickness, 8½ in. ;

high flood level, 58.00 (assumed the same as that at the railway bridge) : free-board, 5 ft over high flood level to the bottom of the T-beam at the centre of the span : maximum discharge, 6,63,548



cusecs observed on the 6th December 1946 at the weir. The catchment area is 5000 sq. miles.

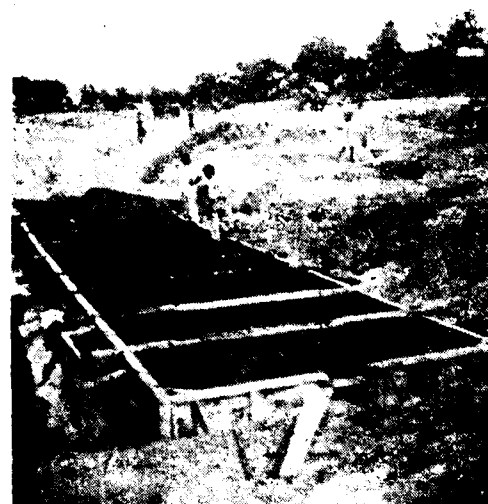
Costs

The bridge, work on which is in progress, is expected to cost Rs 8,23,000 for the foundations, Rs 2,95,700 for the substructure and Rs 10,77,000 for the superstructure. The approaches will cost about Rs 2,00,000. The total cost of the work is estimated to be about Rs 25,00,000.

The expected rate per lineal foot is estimated to be Rs 910.5, and per square foot of roadway excluding foot-paths Rs 41.4 and including footpaths Rs 28.5.

Foundations

Pier Nos. 1 to 21 where the substratum is clay are founded on wells with three separate wells under each pier. Each well is



Reinforcement for Well Cap (Pier No. 1)

to be 7.3 tons per sq. ft. A 15 in. thick continuous capping slab of reinforced concrete is laid over the top of the three wells. The remaining 11 piers and the two abutments, under which there is laterite rock, are founded on mass cement concrete (1:3:6) carried into rock to a suitable depth (varying from one foot to two feet).

The piers and abutments, originally intended to be of coursed rubble masonry in 1 : 5 cement mortar, have actually been built in mass concrete (1 : 3 : 6). The piers will be capped with reinforced concrete slabs, 18 in. thick, which serve also as bed blocks under the T-beams.

Superstructure

The superstructure is very simple in design and consists of four T-beams with a 8½-in. deck slab and a 3-in. wearing surface (1:1½:3). The footpaths, raised 2 in. above the level of the road crown, have been designed hollow to save on dead load and to accommodate the services.

The reinforced concrete handrail will stand 2½ ft high above the footpath.

Bearings

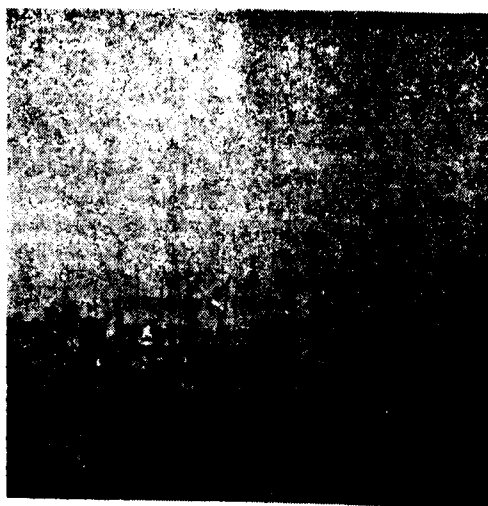
Quotations were invited for roller and rocker bearings and it was found that such bearings under all the 132 beams would cost no less than Rs 2,50,000 that is, almost 20 per cent of the

square in shape (7 ft by 7 ft inside) with rounded corners. The steining walls, in 1 : 3 : 6 cement concrete are 2 ft to 2.5 ft thick with bond rods and plate ties. The wells are plugged at the bottom with 6-ft deep cement concrete plugs (1 : 2 : 4) filled with well-rammed sand, and sealed at the top with 2-ft thick plugs of 1 : 3 : 6 cement concrete. The pressure under the well foundations is calculated

cost of the superstructure. The design was modified to provide sliding plate bearings at the expansion ends and concrete necks at the stationary ends.

Expansion Joints

For a range of temperature of 60 deg. F. and a coefficient of expansion for reinforced concrete of 0.000065 gaps of $\frac{1}{4}$ in. are to be provided between consecutive spans.



Well sinking in Progress.

Execution

Before commencement, work totalling about Rs 20,00,000 was put to tender and the bids which came in ranged from Rs 24,60,000 to Rs 41,30,000. As even the lowest tender (20.1 per cent above the estimate) was outside the permissible margin of 10 per cent, work was taken in hand departmentally.

Later on, well foundation work was given out on contract at 12.3 per cent above the estimated rates. Open foundations have been completed departmentally.

Recently tenders were again invited for the superstructure work and bids ranging from 12.2 to 33.3 per cent above the estimates were received. The work has been let out to the lowest bidder.

This experience has shown that though direct working may not always be as economical as work done on contract, **readiness to execute works departmentally helps to keep the contractor's bids down.**

Present stage of the construction

According to the latest reports, the two abutments and all the piers in the shorter spans have been completed. In the longer spans, well foundations have been completed under six piers and are in progress under other piers. Superstructure has not yet started. The work which commenced in February 1950 is progressing according to schedule and is expected to be completed in June 1953.

3. THE KATHJURI BRIDGE

The river Mahanadi traverses very hilly country throughout the upper part of its catchment of more than 50,000 square miles. Immediately after it debouches into the plains through narrow (2,800 ft wide, gorge at Naraj, it bifurcates into two arms called the Mahanadi and the Kathjuri. Further down each of these arms, in its turn, bifurcates and throws out two more arms, the Birupa and the Kuakhai, and all these arms branch out again and again to form the Mahanadi delta.

Cuttack, the "old" capital of Orissa, is situated at the head of the delta between the first named arms of the river, the Mahanadi and the Kathjuri. During the monsoon, when the rivers on each side of the town are in flood, direct road communications used to be cut off from all sides and access was obtained by rail or power ferry.

To-day the Kathjuri arm is bridged and a temporary roadway is available for seven months in the year under the anicut on the Mahanadi arm of the river. These improvements have been carried out with National Highway funds.

Looking at the map it is easy to visualise the Mahanadi as a huge monster, with its weak and powerless victim, Cuttack, in its jaws. A constant struggle has been going on between the two for centuries. Although it is impossible to extricate the victim entirely, engineers have managed to keep it alive. Bunds, dams, and dykes prevent the monster from closing its jaws and swallowing up the victim, and two lifelines have been thrown in to keep it going and rescue it gradually.

One such lifeline is the railway built at the end of the last century. The road was there even before the railway, but recently two large bridges have been built to make the roadway to the south over the Kathjuri and the Kuakhai effective as a lifeline. These bridges have enabled the State to start life afresh with its "new" capital at Bhubaneshwar.

Cuttack, from all accounts, has perhaps the worst crowded "bustee", of all the State capitals in India. Hedged in by the river, it has no room to expand. Always menaced by floods, the people and the Government might well hesitate to build for the future. Large parts of the town are as much as 14 ft below the flood level of the river. It is not surprising that the place is congested, badly built, unhealthy, and malarious. But with all its defects it is a large town and will be there for a long time to come. It must have communications with the outer world.

THE KATHJURI BRIDGE

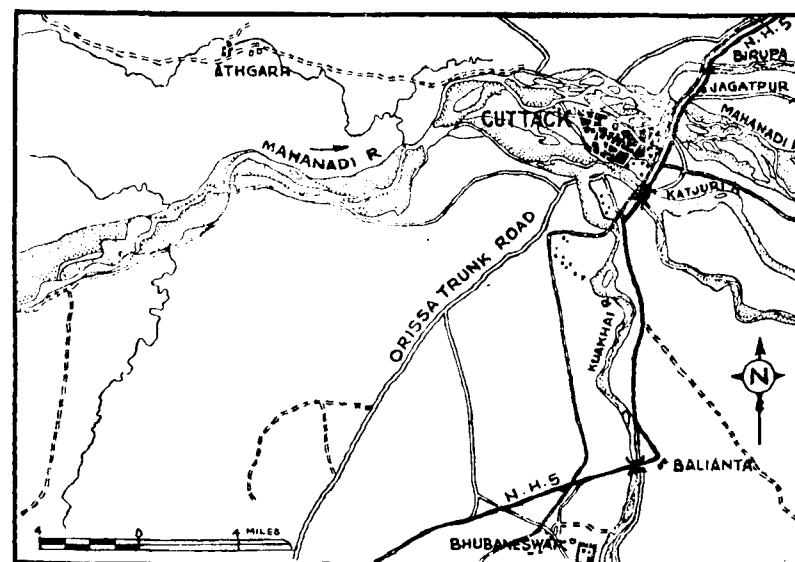


(A view of the completed Bridge).

Four Bridges

To achieve this purpose at least four bridges will be required on the four branches of the river. The Kathjuri and Kuakhai bridges have been built first to link up the new and the old capitals.

The Kathjuri bridge is sited 800 ft downstream of the railway bridge. This site was selected so that the road should not have to cross the railway track twice and so that the approaches should run outside the congested part of the town and at the same time avoid the unstable bank area upstream of the railway bridge.



The superstructure of the bridge consists of 1:2:4 vibrated reinforced concrete balanced cantilever and suspended spans, supported on piers and abutments made of 1:3:6 mass concrete, lightly reinforced and founded on wells.

The main dimensions and other particulars of the bridge are:

Overall length face to face of abutments, 2828 ft; 17 spans, 160 ft. each; 2 end-spans, 54 ft each; height of pier shaft from plinth level to level of bearing seat, 32.5 ft; width of pier shaft at plinth level, 9 ft 7 in.; length of pier shaft at plinth level 36 ft; batter in piers, 1 in 24.

Live load, Indian Roads Congress Class AA on the carriage-way : live load on the footpath, according to Clause 207 of the Indian Roads Congress Bridge Code :

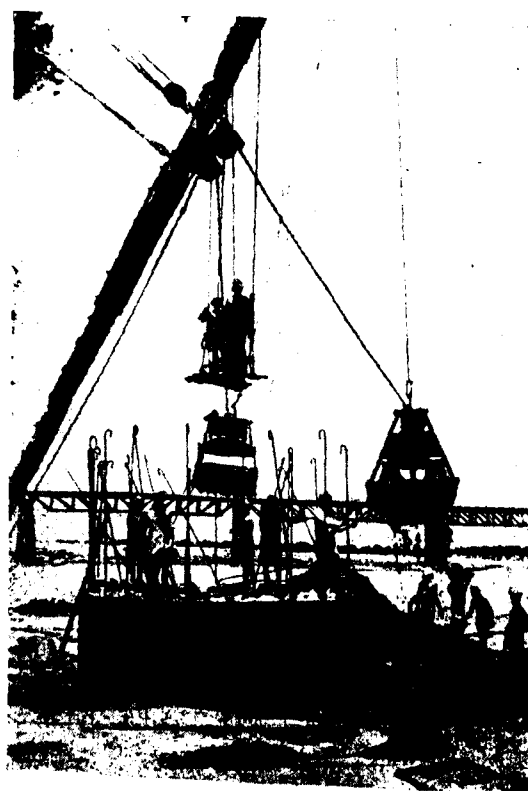
Average pressure intensity on foundations under piers at R.L. minus 35.00, 7 tons per sq. ft.; under abutments, one ton per sq. ft. :

Temperature variations ± 30 deg. F.

Average velocity of water under the bridge, 5 ft per second : velocity under certain piers, 8 ft per second : discharge, 577,000 cusecs.

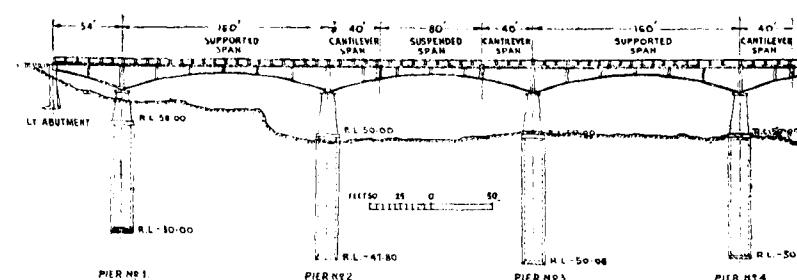
Costs

The total cost of the bridge was Rs 45,89,000/-. The foundations cost Rs 20,36,000 -, piers and abutments Rs 3,13,000 -, bearings Rs 1,30,000 and superstructure Rs 17,60,000/-.

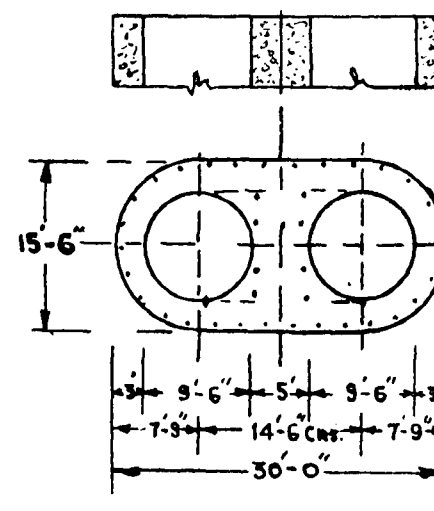


Dredging in Progress.

THE KATHJURI BRIDGE



Part Longitudinal Elevation



Plan & Section of well.

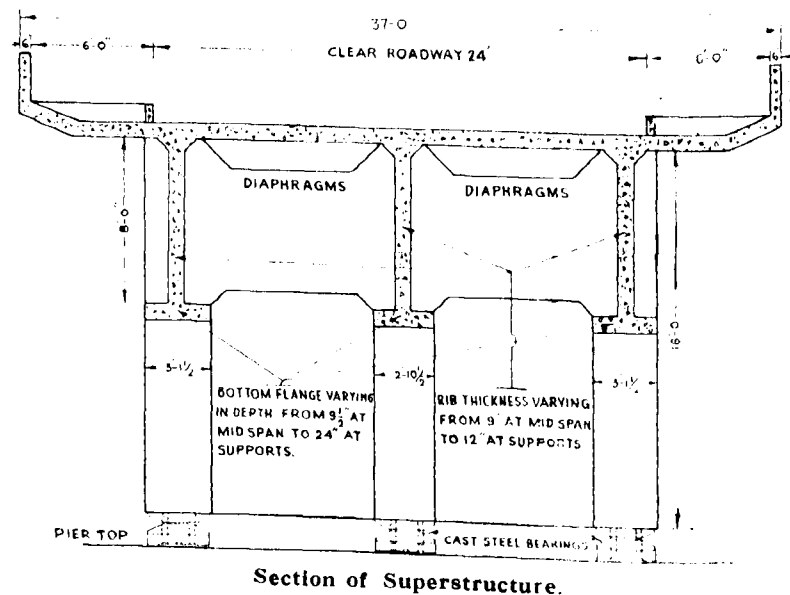
The cost per running foot of the bridge was Rs 1,623 ; per square foot of carriage-way (excluding footpaths), Rs 67 ; per square foot of roadway (including foot-paths), Rs 45/-. The cost per square foot of the entire elevation (that is, the area enclosed between the line connecting the uppermost points of the superstructure and the line connecting the lowest parts of the foundations) was Rs 102/-.

Foundations

Each pier was founded on a twin well sunk to the approved depth, so as to rest on stiff black clay. The well had an overall size of 30 ft by 15 ft 6 in. and was of lightly reinforced concrete (1 : 3 : 6).

Superstructure

The superstructure consisted of three reinforced concrete girders spaced 11 ft centre to centre, with a 9 in. thick reinforced concrete deck slab, covered with a 3 in. wearing coat of 1 : 2 : 4 cement concrete. The girders were 18 ft deep at the support and 8 ft at the centre, and were of a web and flange type with a web thickness of 9 inches.



Roller and Rocker bearings were provided on the piers and sliding plate bearings at the free end under each of the three girders of the suspended spans. Each roller or rocker bearing was designed for a load of 400 tons.



Concreting the Superstructure.

Well Sinking Difficulties

The engineers responsible for the construction were faced with numerous problems in the course of sinking wells for the foundations. Many wells went out of plumb and shifted from their correct positions. They were corrected by regulating the dredging, pulling and pushing into position during sinking, and eccentric or cantilever loading. These methods were not completely successful in every case.

The feature which caused special headache on this work was that in some of the wells Artesian springs were encountered. The water level in these wells rose and water began to overflow at a tremendous rate. The wells began to sink abruptly under their own weight. A detailed description of the devices adopted to overcome these difficulties is given in the Journal Vol XIV-2.

Test Loads

The wells where artesian conditions had been met with, were load tested. The test loads which ranged from 1,000 tons to 1,200 tons on a twin well, were so calculated as to give the same unit pressure under the steining as would develop under the whole area of the well after the bridge came into use.



Eccentric Loading.

Superstitions

During the construction of this bridge the usual rumour that at such big bridge works the river god is propitiated by a human sacrifice took hold of the populace and led to a minor riot. The Government had to issue a press note to dispel the impression from the public mind.

Work on this bridge commenced on 1-11-1947 and the bridge was opened to traffic on 31-8-1951.

4. THE CAUVERY BRIDGE AT PUGALUR

With the opening of the bridge over the Cauvery at Pugalur in Madras in December 1951, dreams more than two decades old came true. As long ago as 1936 Vipan* included a bridge at Pugalur in his scheme. But before that scheme could be put into force, the Second World War intervened. Then, fortunately, came the National Highway Scheme and the alignment of National Highway No. 7 from Banaras via Rewa, Jabalpur, Nagpur, Hyderabad, Bangalore, and Madura on to Cape Comorin passed over the Cauvery in this neighbourhood. A bridge already badly needed, became all the more necessary.

The river Cauvery divides two very prosperous districts of Madras, viz., Salem and Trichinopoly, and is unbridged in this area for almost a hundred miles of its course. Before the bridge at Pugalur was built there were only two bridges over this river, one at Trichinopoly and the other about 100 miles upstream at Bhavani. That at least one more intermediate bridge was needed immediately was obvious, but it was not equally obvious where this bridge should be sited. For years a controversy raged about the actual location of the bridge. No less than five different sites were considered and their merits and demerits debated in detail. Finally the issue narrowed down to the two sites at Nanniyur and Pugalur.

The Government of India, who had to pay for the work, favoured the Nanniyur site because the road along that route was practically the shortest distance between the obligatory points of Karur and Namakkal. Moreover, the river at that site was narrower than at Pugalur. The adoption of the Pugalur site was expected to cost Rs 6 lakhs more than the alignment via Nanniyur. But Madras wanted the bridge at Pugalur for various reasons. Finally, as a compromise, the Central and State Governments agreed to split equally the excess expenditure to be incurred by adopting the more costly route via Pugalur. According to this arrangement the Centre paid Rs 27 lakhs out of the total estimated expenditure of Rs 30 lakhs.

The bridge was designed as a reinforced concrete structure with continuous Tee-beams supported on piers and abutments

* "Scheme of Road Development for the Madras Presidency" by A. Vipan, M. Inst. C.E., Special Engineer, Road Development, Madras (1935), page 32.

THE CAUVERY BRIDGE AT PUGALUR

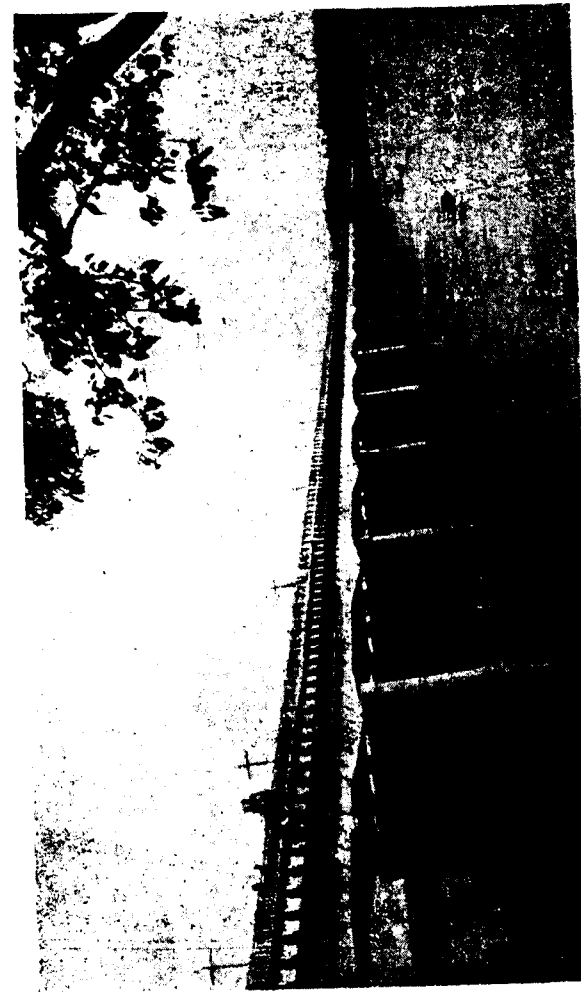


Photo No. 1—The Completed Bridge.

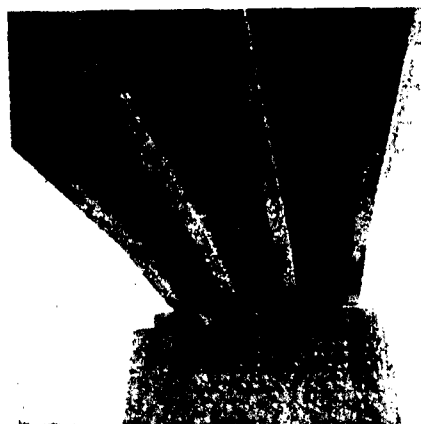


Photo No. 2—T-Beams and stiffeners

of coursed rubble masonry in cement mortar. The continuous girder type was selected as rock foundations at a reasonable depth were found under most of the piers. There were 12 units, each continuous over three spans and one unit of four spans in the middle. Construction started on the 9th May 1949.

The main dimensions of the bridge are:

Length between abutments, 2634 ft; 40 equal spans of 60 ft clear at the top of the piers; pier thickness, 6 ft at the top; batter: 1 in 12 for abutments and 1 in 24 for piers; height of piers about 20 ft;

Roadway, 22 ft between kerbs;

Depth of Tee-beam rib, 6½ ft at support and 5 ft at midspan; thickness of rib, 15 inches; 12 in. x 12 in. cross-beam for stiffness, 10 in each span;

Thickness of slab, 7½ inches;

Wearing surface, 1 : 1½ : 3 cement concrete 3 in. thick;

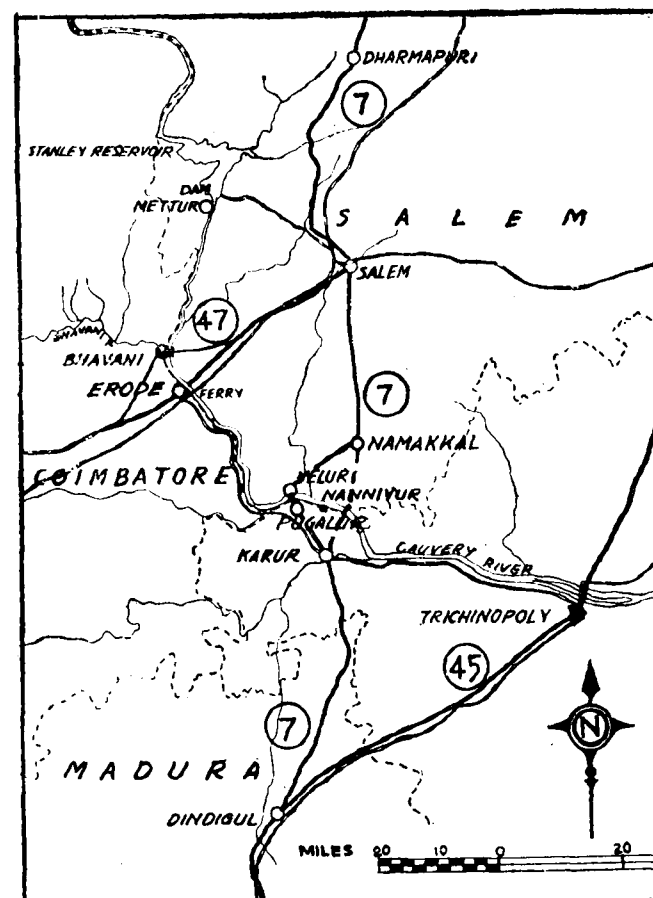
Total depth of well sinking, 2256 ft; average depth of well, 36 ft; maximum depth, 40 ft;

Expected maximum discharge under the bridge, 2,86,100 cusecs.

Costs

Total cost including approaches, Rs 25.87 lakhs; cost of sub-structure, Rs 8,09,000; cost of piers, Rs 2,81,000; cost of super-structure, Rs 11,40,000; cost of approaches, Rs 1,95,000.

The cost per running foot of the bridge is Rs 965; per square foot of carriageway, Rs 44.5; cost per square foot of the entire elevation (that is, the area enclosed between the line connecting



the uppermost point of the superstructure and the line connecting the lowest parts of the foundations) is Rs 20.

Mettur Dam Affects Design

A design finalized in 1940 contemplated building 48 spans of 60 ft each, that is, a bridge about 3,200 ft long catering for the highest known discharge of 4,85,500 cusecs. This flood had passed down the river at the bridge site in 1924, the year of phenomenal floods in Southern India. But, since the completion of the Mettur Dam in 1934, the discharge in the Cauvery below the dam was fairly controlled. In 1940, the knowledge of the extent of this control was yet incomplete and the bridge was

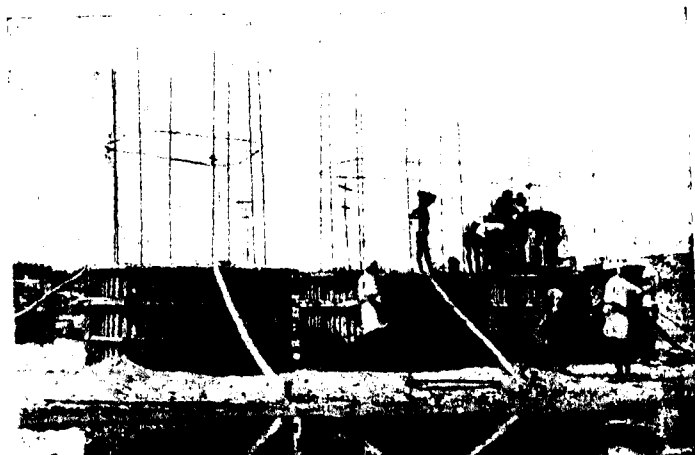


Photo No. 3—Wells.

designed for the full river width. But by 1949 the flood absorbing effect of the dam was known fairly well. The maximum discharge let down in the Cauvery at Mettur after 1934 had been only 1,64,000 cusecs. With the maximum discharge of the Bhavani added to it (1,22,000 cusecs), the flood in the Cauvery at the bridge site could not be expected to exceed 2,86,000 cusecs. The length of the bridge was, therefore, reduced from 48 to 40 spans and this gave a saving of about Rs 6 lakhs.

Foundations and Piers

For the left abutment and piers 1 to 8 and 10, where rock was available at shallow depths, open foundations were adopted. The right abutment and the remaining 30 piers were founded on wells.

The piers were built of first class coursed rubble masonry in 1 : 4 cement mortar below low water level, and in 1 : 5 mortar above that level. But in piers 4 to 8 and 10, where heavy springs were encountered, the substructure below low water level was mass cement concrete (1 : 3 : 6).

Elsewhere the well foundations under each pier consisted of three separate rectangular, lightly-reinforced cement concrete wells with rounded corners (Photo No. 3). The outer size of a well was 8 ft × 9 ft (8 ft × 9 ft 6 in. under the right abutment) with a 2-ft thick steining. The three wells under each pier were capped with continuous reinforced concrete slab 32 ft 6 in. × 8 ft 6 in. × 1 ft 6 inches,

Bearings

Eighteen inch thick reinforced concrete bed blocks placed on the piers carried cast steel roller and rocker bearings.

In each 3-span unit the expansion bearings at the end supports were 2-roller assemblies, one intermediate support had a 4-roller assembly, and the other a fixed rocker bearing. Rollers of the same diameter were not available for all the bearings. Three different diameters were to be had, viz., 6½ in., 7 in., and 7¾ in. This difference in size was put to good use in raising the level of the two intermediate bearings and thereby providing camber in the surface slab.

An expansion joint was provided in the deck slab at the end of every unit.

Anchoring Wells

According to the approved specifications the foundations of the piers were to be taken at least 2 ft into rock. This was done as far as possible. In a few cases, however, some wells could not be embedded into rock and rested on sloping strata. Such wells were anchored to the foundation rock by means of 1 in. steel bar dowels (one dowel for every 2 sq. ft. of dredge hole area) embedded 3½ ft into the rock and extending 3½ ft into the well.

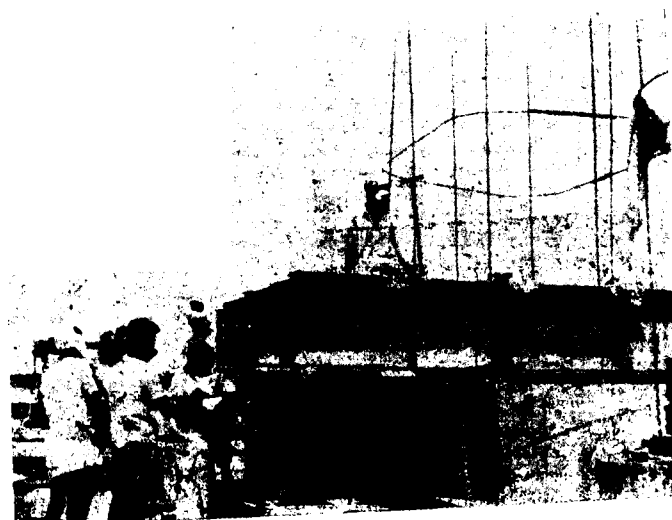


Photo No. 4—Lifting a well.

As boring anchor holes into rock under deep water was a difficult operation, the wells on reaching rock were plugged with about 2 ft of cement concrete and de-watered. The holes for the dowels were then drilled through the concrete plugs so as to extend 3½ ft into rock.

Dowel bars were placed in the holes and a rich cement mortar was packed round them to prevent rusting.

Transplanting Wells

Wells built for one pier as explained below, were shifted bodily and used under another pier. According to the design, pier No. 10 was to have had open foundations, but as de-watering was troublesome and costly the contractors obtained permission to adopt well foundations. The well had gone down hardly 6 ft when rock with a steep side slope was encountered. Fortunately, work on the wells for pier No. 11 had not commenced. The contractors decided to transplant all the three wells to that pier.

Wire ropes were passed under the curb and their ends tied to two rolled steel joists placed at the top of the wells, which were pulled out by means of four 10-ton jacks (Photo No. 4). When the curbs were clear of the ground level the wells were dragged by



Photo No. 5—A well travels to the next pier.

means of winches to the next pier on 1½ in. round steel bars used as rollers (Photo No. 5). Each well weighed about 24 tons. The whole operation cost only Rs 750/-.

Suspended Centering Saves 8 Months

When the working season in 1951 came to an end, water from the Mettur Reservoir was let into the river for irrigation.

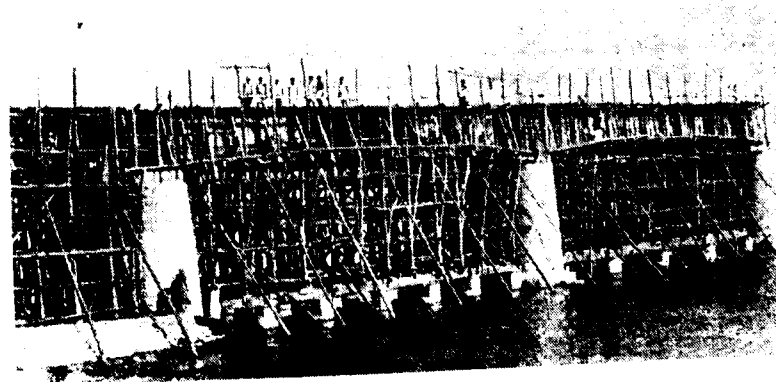


Photo No. 6.

Ordinary timber staging and centering completely obstructs the channel and cannot be used in the flood season.



Photo No. 7.

Special centering leaves the river unobstructed and work can go on in all seasons.

The superstructure for 10 spans still remained to be built. Postponement of the work to the next season would have meant the loss of eight valuable months. Parts of a 120 ft span Callender-Hamilton bridge were available and they were used to build eight continuous trusses, two for each Tee-beam. The outer two trusses were erected on the top of the noses of the piers of a completed unit and rolled out into position. The other six trusses were erected on the deck of the last completed unit and rolled out on timber "horses" and then lowered into position.

The use of this centering saved a whole working season. The bridge was opened to traffic on the 23rd December 1951.

5. THE KUAKHAI BRIDGE

The road bridge over the Kuakhai river is one of the four projected major bridges intended to develop the Mahanadi delta. It is also one of the numerous bridges which will have ultimately to be built on National Highway No. 5 between Calcutta and Madras. It achieved "top priority" for local reasons, viz., the development of the new capital city of Orissa at Bhubaneswar and the need for connecting it with the old capital Cuttack, by means of an all-weather road. Further details of the history of this project were given in the article on the Kathjuri Bridge* on page 419.

A Variable River

The Kuakhai is one of the deltaic branches of the Mahanadi and discharge passing down these branches varies from year to year. Sometimes only one branch takes the main discharge and a few years later this branch gets sluggish and tends to silt up while another branch becomes active. On comparing surveys and other records dating back to 1847 with the surveys made in 1896-97, railway engineers of those days came to the conclusion that "the whole tendency of the river appears to be to convert the Kuakhai into the main channel....." The observed recorded discharge of the Kuakhai was 3,40,000 cusecs in 1896. As if to falsify these prophets, the highest recorded discharge after 1897 was only 2,53,000 cusecs in 1937.

The bed of the Kuakhai has now tended to rise at its mouth and was in 1947 as much as 17 ft higher at that place than that of the Kathjuri at its corresponding point of take-off from the Mahanadi.

These changes in the regime of the river aggravated considerably the normal uncertainties of the situation and made the selection of a suitable bridge site difficult.

Various experts favoured the construction of the bridge at different locations on the river spread over a range of about 15 miles. At each of these locations there was further divergence of opinion about the most suitable exact site. A joint inspection by experts in March 1947 followed by model experiments at the testing station at Poona solved this thorny problem and the present bridge site near Baliana was finally settled.†

Like the Kathjuri bridge, the Kuakhai bridge was built as a reinforced concrete balanced cantilever structure with suspended

* For technical details of the Kathjuri Bridge, see I.R.C. Journal Vols. XIV-2 & XIV-4. For those of the Kuakhai Bridge, see I.R.C. Journal Vols. XV-3 & XV-4.

† Please see map on page 421.

THE KUAKHAI BRIDGE



A view of the Completed bridge.

spans in alternate bays. The span was, however, much smaller on this bridge.

Dimensions

The main dimensions of the bridge are :

Overall length face to face of abutments, 1696 ft ; 15 spans, 106 ft each, and 2 end spans, 53 ft each ; height of pier shaft from plinth level to level of bearing seat, 18.5 ft ; thickness of pier shaft at plinth level, 8 ft ; batter in piers, 1 in 16.

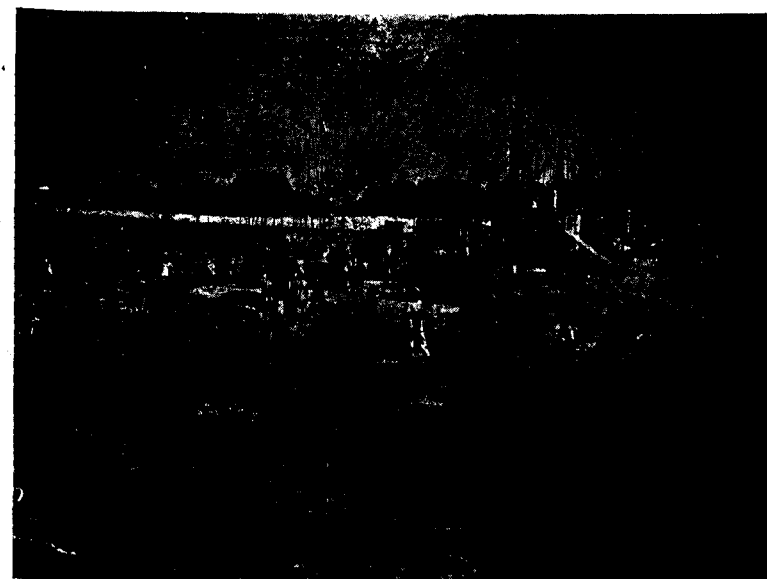
Carriageway, 24 ft ; 2 footpaths, 6 ft each ; depth of girders, 13 ft 6 ins. at the supports, and 6 ft at the centre ; web thickness 9 inches.

Live load, I.R.C. Class AA on the carriageway ; live load on the footpath, according to Clause 207 of the I.R.C. Bridge Code.

Maximum pressure intensity on foundations under piers 7 tons per sq. ft.

Temperature variation, ± 30 deg. F.

Average velocity of water under the bridge, 5 ft per second ; discharge, 2,60,000 cusecs. (This calculated maximum discharge takes account of the dam across the Mahanadi under construction at Hirakud some 200 miles upstream of the Kuakhai bridge site).



The start of the work.

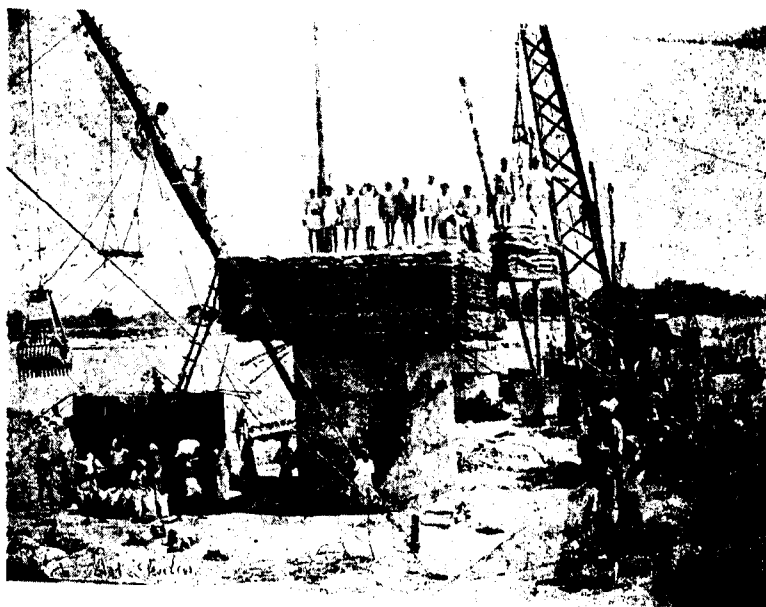
Well Foundations

One of the main differences between the Kathjuri and Kuakhai bridges was that in the latter two separate circular wells were used under each pier instead of a twin well as at the Kathjuri. This made sinking easier in the much stiffer clay existing at the Kuakhai site.

Of the 32 wells forming the foundations of this bridge, only 7 sank absolutely straight. The shift and tilt in the remaining 25 wells varied from $\frac{1}{4}$ in. to $1\frac{1}{2}$ in. in the worst cases. This was considered within the range of permissible tolerance and the piers were built in the designed positions.

The wells were 13 ft 6 in. in diameter (outer) with a 2 ft 6 in. thick steining. After sinking and plugging they were connected at the top with 3 ft thick vibrated reinforced concrete well caps.

The piers were built on lightly reinforced 1:3:6 cement concrete with plums. The Abutments were designed as reinforced concrete retaining walls for earth pressure and traffic loads on the fill behind but not for taking any load from the bridge.



Sinking devices: Kentledge loading on one well and dredging in progress in another

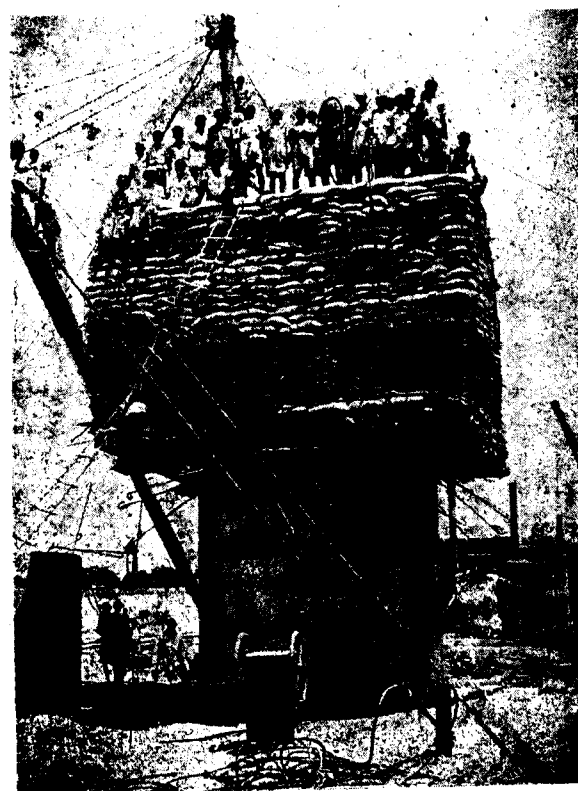
The joints in the main reinforcement in the beams were welded.

The roller and rocker bearings were designed for a load of 240 tons (400 tons on the Kathjuri bridge).

Costs

The bridge has cost the Central Government a total amount of Rs 23,21,000. Of this, the foundations cost Rs 8,12,000; piers and abutments Rs 1,34,000; and superstructure Rs 9,69,000.

Cost per running foot of the bridge, Rs 1368; per sq. foot of carriageway, Rs 57; per sq. foot of roadway, Rs 38/7; per sq. foot of entire elevation (i.e., the area enclosed between the line connecting the uppermost points of the superstructure and the line containing the lowest parts of the foundation) Rs 136/-. Considering the depth to which the wells were sunk, the cost of the work may be looked upon as normal or even below normal.

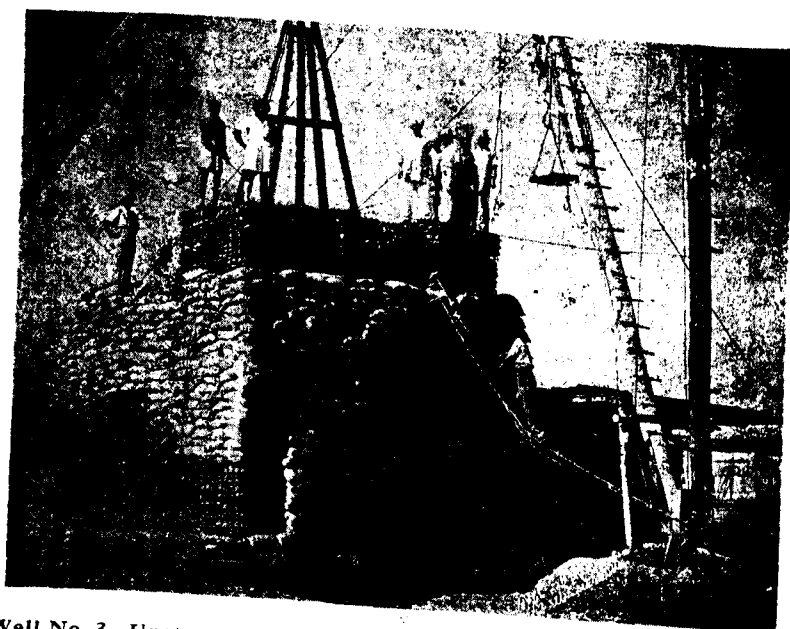


A well loaded with kentledge

Tests

Before the superstructure was taken in hand, test cubes were made to ascertain the strength of concrete from different sources. Hard granite stone metal and gravel were found suitable. Selected broken gravel was, however, also allowed in the superstructure mixed with 50 to 60 per cent of broken granite.

During the actual construction, 2 to 3 test specimens were taken every day from the mix used in the work and later tested to see whether the ultimate strength conformed to specifications. The strength of 1:2:4 cement concrete at 28 days ranged from 3,257 lb to 5,413 lb per square inch.*



Well No. 3—Upstream. When found excessively difficult to sink even with sufficient Kentledge, special chisel cutter was used

Sand was tested for bulking every day of concreting. Percentage reduction ranged from 22 to 27 and an adequate proportion of sand was used to compensate for this bulking.

The metal used in the roller and rocker bearings was tested both physically and chemically.*

The bridge was load-tested after completion under dead loads calculated to give rise to stresses the bridge had been designed for. The test was successful and the spans subjected to the test recovered completely when the load was removed.

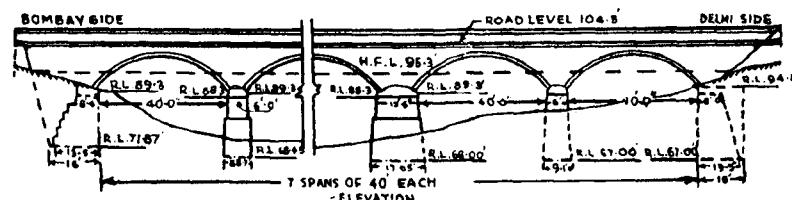
Work on the bridge began in November 1947 and it was thrown open to traffic in January 1951.

* Details of the tests are given in Paper No. 155 published in Vol. XV—8.

6. THE TANSA BRIDGE

Forty-six miles north of Bombay, near Ambadi in the district of Thana, Bombay State, the route from Bombay to Delhi via Gujarat and Rajasthan, called National Highway No. 8, crosses the river Tansa over the recently constructed Tansa Bridge, which was thrown open to traffic in May 1951 by the Hon'ble Shri Jivraj N. Mehta, Minister for Public Works, Government of Bombay.

The bridge replaces the vented low causeway which existed about 600 feet upstream. This causeway was only 18 feet wide between the wheel-guards and rose only 6 feet above the lowest water level in the stream. It did not satisfy the needs of traffic and its frequent submergence during the monsoon often interrupted traffic. Besides, it needed frequent repairs due to scour on its downstream side.



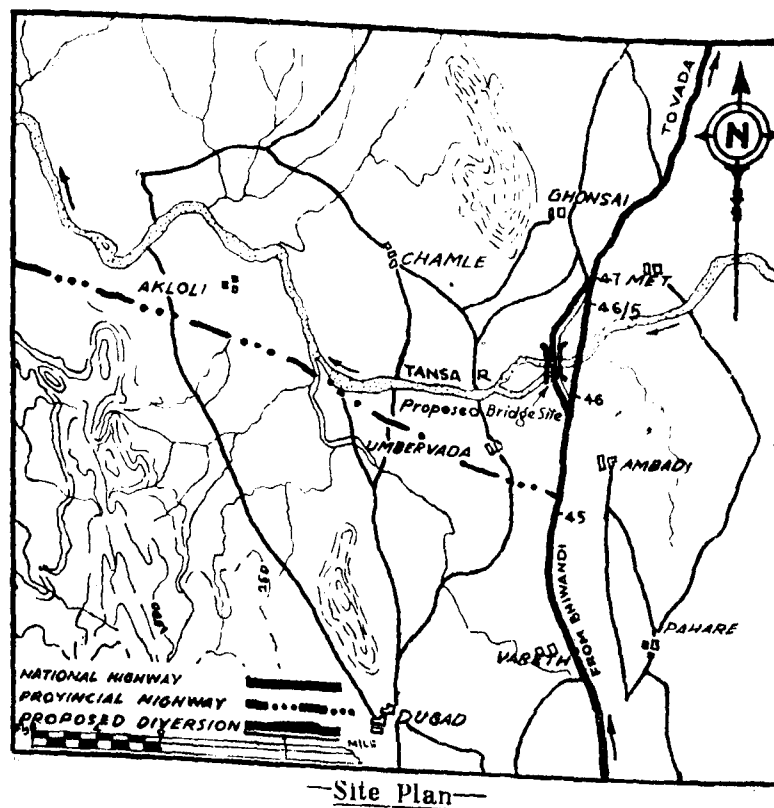
decided to do it departmentally, since the rates tendered by contractors were excessive. It took about two years and a half to complete the bridge.

The Site of The Bridge

About 600 feet downstream of the existing causeway, a site was available where the river had well-defined banks and flowed through a narrow gorge. Here the length of the bridge required was relatively small and the structure could be founded on rock which existed not much below the surface of the bed of the stream. The site was, therefore, approved. This site entailed the construction of new approaches about a mile long altogether.

The Foundations

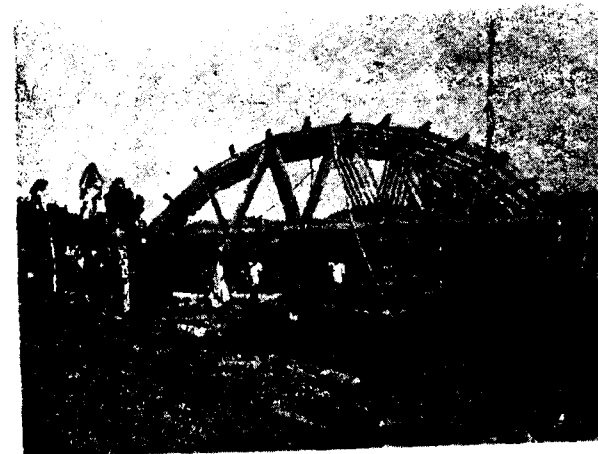
Open type foundations were designed and laid without any difficulty. The overlying weathered rock strata had to be re-



moved to an average depth of 5 ft 9 in. to reach hard rock. The cement concrete mix used in the foundations was 1 : 3 : 6.

The Superstructure

Since hard rock was available for the foundations and the section of the stream was deep and narrow, it was possible to adopt arched spans. Forty foot clear spans were found economical. Unreinforced arches were designed with a view to avoiding the use of steel of which there was a scarcity in the country. The loading applied in the design was the I.R.C. Class A2 lanes, or Class AA one lane.



Steel centring for arches

At first it was intended to design spandrel-filled segmental arches, 30 ft span, in lime concrete in view of the fact that lime concrete is resilient. But the idea was given up as good lime was not available within economical distance of the site of the work. It was finally decided to use pre-cast cement concrete blocks (1 : 2½ : 5) in preference to stone or bricks, as the pre-cast cement concrete blocks have greater compression strength than stone or brick, and the arches constructed with them are capable of adjusting themselves to the stresses induced by shrinkage and changes in temperature better than the cast-in-situ cement concrete.

Salient Features

Spans, 7 of 40 feet each; length of the bridge from abutment to abutment, 345 feet; thickness of the arch ring, 2 feet throughout; rise of the arch, 1/4 span; thickness of the cushion

at the crown of the arch, 2 ft 6 in; piers, 6 feet wide at the top with a suitable batter on each side; abutment, 8 feet wide at the top with a back batter 1 in 4; abutment pier 2 Nos, 12 feet 6 in. wide at the top with a suitable batter on each side: pier caps, 1 ft thick cement concrete.

Abutments and piers were constructed of second class coursed rubble stone masonry in 1 : 6 cement mortar.



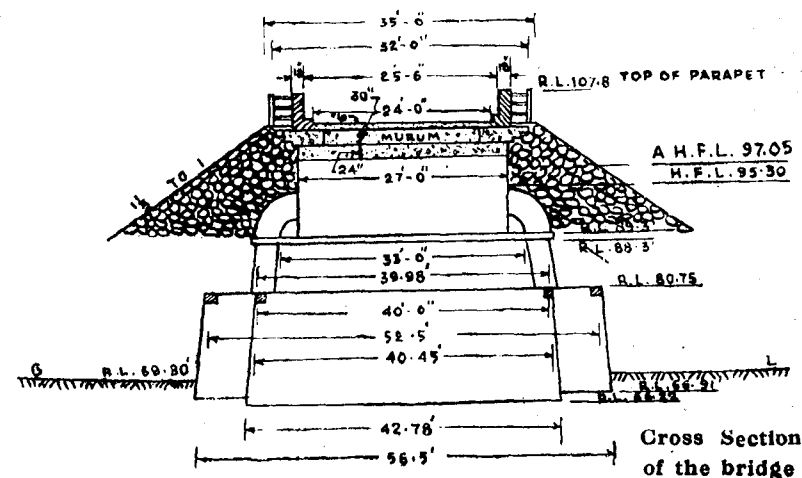
View showing the work in progress.

Width of carriageway, 24 ft between wheel-guards; wearing surface, 6 in. thick cement concrete slab (1 : 2 : 4).



Another View showing the work in progress.

The Tansa drains a catchment area of about 150 square miles and the bridge was designed for a discharge of 84,500 cusecs flowing with a velocity of 30 feet per second.



Cost

The original estimate of the cost framed in the beginning of 1948 was Rs 2.87 lakhs. The actual completion cost of the work was Rs 4.33 lakhs. This rather high excess of the actual expenditure over the estimated cost was mainly due to the rise in the cost of materials during the period of construction. Per running foot the actual cost works out to Rs 1,015 excluding the cost of the approaches.



Piers under construction

ture over the estimated cost was mainly due to the rise in the cost of materials during the period of construction. Per running foot the actual cost works out to Rs 1,015 excluding the cost of the approaches.

7. THE CHAMPAVATI BRIDGE

The river Champavati is one of the thirteen major rivers that cross National Highway No. 5 linking Madras and Calcutta. At the bridge site its catchment area is about 520 sq. miles and, because of its swift current during the monsoon and the shallows existing in its bed during the dry weather, it has always presented difficulties to ferry traffic.

The necessity for a bridge to span this river was felt as early as 1931 when a screw-pile bridge of the through Pratt girder type with eleven 60-ft spans was built at a constricted site near Nathavalsala about 508 miles from Madras. Unfortunately, this bridge was washed away the same year due to unprecedented floods which followed breaches in a series of tanks and raised the maximum high flood level for which the bridge had been designed from R. L. 87.75 to 102.42.

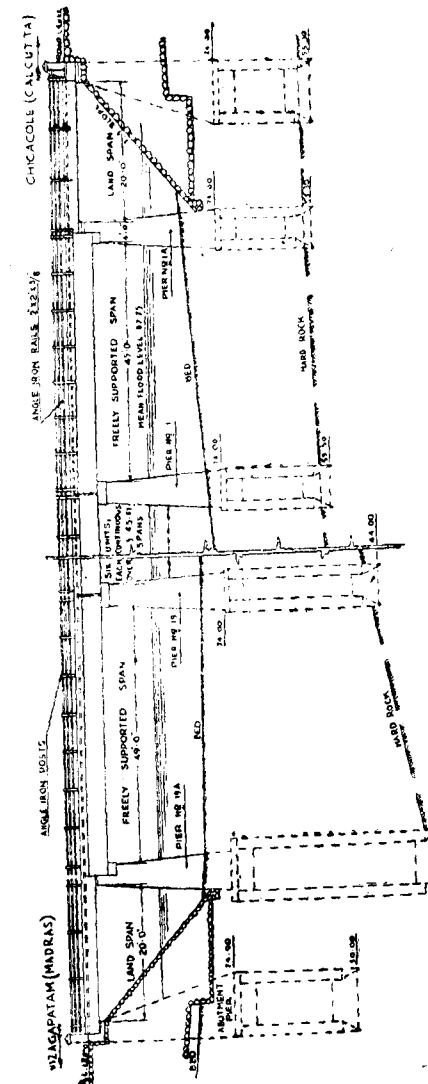
Eleven years later, i.e., in 1942 another attempt was made to bridge this river at a point about 1,600 ft downstream of the previous site. Twenty spans each of 45-ft reinforced concrete T-beams and slabs were proposed. The construction of the bridge, financed from the Road Development Fund, was undertaken by the Vizagapatam District Board. Owing to a dispute between the contractors and the District Board the work had to be abandoned after the wells for the righthand side abutment and seven piers had been partially sunk.

It was not till 1951 that work on this bridge was re-started as a part of the first 5-year road-development programme.

The Site

After thorough investigation it was decided that the new bridge should be built at the site of the unfinished bridge which lies on the approved alignment of National Highway No. 5. Here the river has wider banks and hence a comparatively lower velocity of flow than at the site selected in 1931. Besides, some saving will be effected by making use of part of the work executed during 1942.

The maximum flood level reached during 1931 was due to exceptional circumstances not likely to be repeated. To design a bridge for this maximum flood would entail the construction of very high and expensive approaches resulting in the locking-up capital to insure against a remote risk. The ventway of the new bridge as now designed will be sufficient to pass a flood discharge corresponding to a maximum flood level of 87.75, the maximum recorded before and since 1931, but a free-board of 9.5 ft has been



Longitudinal Section of the Champavati Bridge.

allowed as a margin instead of the usual 3 feet and the approaches have been designed to be safe against overtopping by floods.

Foundations

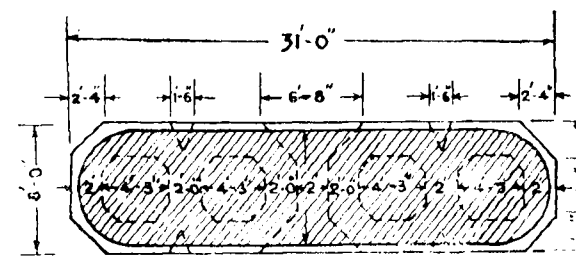
Nature of the soil at the bridge site is sandy, but hard rock is met with at depths varying from 18 to 33 ft below the bed with an intermediate layer of clay. Well foundations for all the piers and the abutment pier on the Chicacole side have been taken at least 2 ft into the hard rock. Where this was not possible, 1-in. diameter anchor rods were embedded at least 3 ft 6 in. deep into the hard rock below the bottom plug. The abutment pier on the Vizagapatam side has its wells resting on hard clay.

The abutment-piers have two twin octagonal wells each 12 ft $\times$ 14 ft 6 in. (outer dimensions) with a gap of 2 ft in between the wells, whereas each pier has two twin octagonal wells each 14 ft 6 in. $\times$ 8 ft (outer dimensions).

The wells have cement concrete (1 : 3 : 6) steinings 2 ft thick with bond rods and flat ties, and R. C. C. (1 : 2 : 4) curbs 3 ft deep and 2 ft thick at the top with cutting edges of mild steel T-sections 6 in. $\times$ 4 in. $\times$ 2 in. The inside of the wells were plugged with 5 ft cement concrete (1 : 2 : 4) at the bottom and with 3 ft cement concrete (1 : 4 : 8) at the top, the space in between being



Start of Well Construction



PLAN OF FOUNDATION-WELL UNDER PIER.

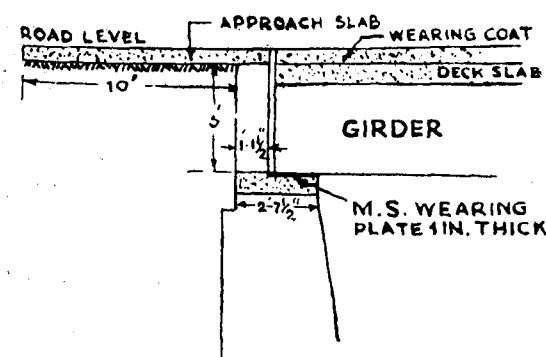
filled with clean river sand. The wells and the abutment piers have a continuous cap 1 ft 6 in. thick reinforced at top and bottom.

Substructure

At first it was proposed to have piers and abutments of coursed rubble masonry in cement mortar. Calculation showed, however, that to keep the tensile stress in the masonry and all the forces induced by traffic and temperature within the permissible limits the section would have to be very massive, necessitating widening of the wells that had been partly sunk in 1942. It was finally decided, therefore, to use mass cement concrete 1 : 3 : 6 for the abutments and piers.

A reinforced cement concrete (1 : 2 : 4) approach slab will be provided to disperse the live load over the fill behind each abutment.

Superstructure



The superstructure consists of reinforced concrete T-beams and deck slab. The bridge, designed for two lanes of "I.R.C. Class A" loading or one lane of "I.R.C. Class AA" loading, has six units, each continuous over 3 spans

and two simply supported units, one at each end. In addition, there are two land-spans. Holes will be left in the deck slab to allow the entrapped air to escape, should the bridge be submerged at any time in the future.

Bearings

Three of the supports of each continuous 3-span unit will have sliding plate bearing while the fourth support will be fixed. The simply supported spans will have sliding bearings at one end and fixed bearings at the other.



Piers under Construction

Approaches

On the Vizagapatam side the length of the approach is 348 ft whereas on the Chicacole side the length is 368 feet. The width of formation of the approaches will be 32 ft with side slopes of $1\frac{1}{2}$ to 1.

To guard against breaches if the approaches are submerged during high floods their slopes and berms will be protected with 9 in. stone pitching laid over 3 in. shingle wherever the height of submergence is likely to be more than 3 feet. The approaches will be level for a distance of at least 50 ft on either side of the bridge.

Salient Features

Spans, 19 of 45 ft, one span of 49 ft, and two land spans of 20 ft; lineal waterway, 944 ft; length of the bridge from abutment-pier to abutment-pier, 1033 $\frac{1}{2}$ ft; thickness of the reinforced cement concrete (1 : 2 : 4) deck slab, 7 $\frac{1}{2}$ in.; reinforced cement

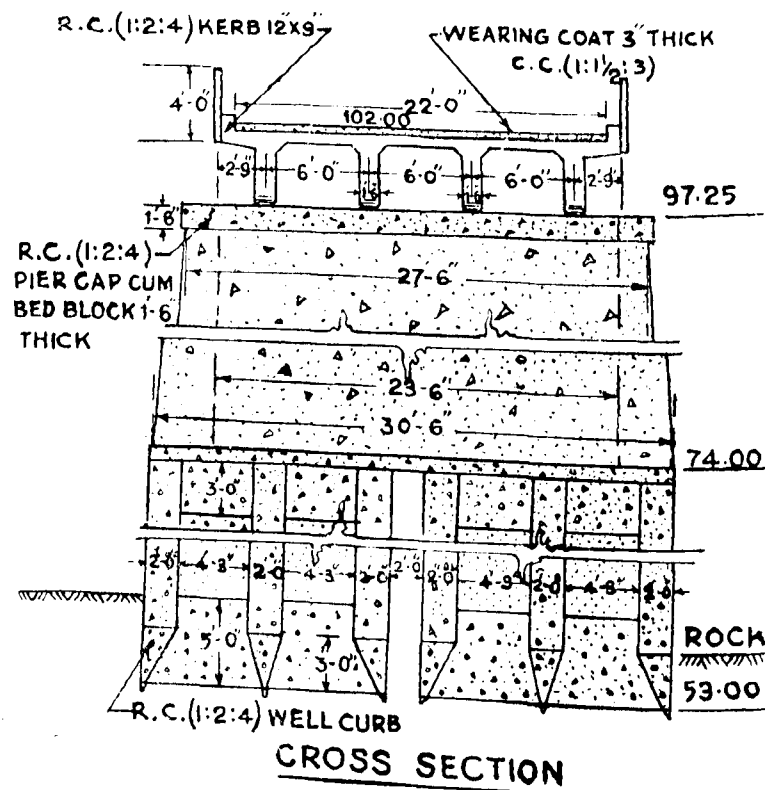


A view showing the bridge under construction



Another View

concrete T-beams, 1 ft 6 in. $\times$ 4 ft 6 in.; piers, 4 ft wide at the top and 7 ft wide at the bottom; abutment-pier, 4 ft wide at the top and 12 ft wide at the bottom.



Width of carriageway, 22 ft between kerbs; R.C.C. (1 : 2 : 4) kerbs, 12 in. $\times$ 9 in.; wearing surface, 3 in. thick cement concrete slab (1 : 1½ : 3).

The bridge has been designed for a discharge of 68,800 cusecs flowing with a velocity of 8½ ft per second. The anticipated afflux is 8 inches.

Cost

The bridge, work on which is still in progress, is estimated to cost Rs 13.2 lakhs. Per running foot the bridge is estimated to cost Rs 1277/-, excluding the cost of the approaches.

8. THE TALDANGA BRIDGE

The Taldanga bridge takes N. H. 5 (Calcutta-Madras Highway) across the Taldanga canal which traverses the town of Cuttack. The construction of the bridge was taken up in May 1950 while the Kathjuri and the Kuakhai bridges were being built, as without bridging the canal these two bridges could not be used effectively to connect Cuttack and Bhubaneshwar, the old and new capitals of Orissa.

The Taldanga Canal takes off from the Mahanadi near Cuttack and is an irrigation-cum-navigation canal.

The selection of site presented no problems in this case as the bridge was to be on an artificial channel.

Numerous Alternative Designs

Numerous alternative designs were examined in detail before deciding upon the design finally adopted. A design for a bow-string girder bridge of 100 ft span with masonry arch spans in the approach was got out in the Central Roads Organization. The firm of contractors who were building the Kathjuri bridge within less than a mile from the Taldanga bridge site was asked to quote. As the quotation was considered high the firm was asked to submit revised quotations for alternative designs. These were :

- (1) R. C. bow-string-girder design
- (2) Uniformly thick R. C. solid slab (103 ft main span) with 40 ft cantilevers.
- (3) R. C. solid slab of varying thickness (103 ft main span) with 40 ft cantilevers.
- (4) R. C. T-beam and slab (106 ft main span) and 38½ ft cantilevers.

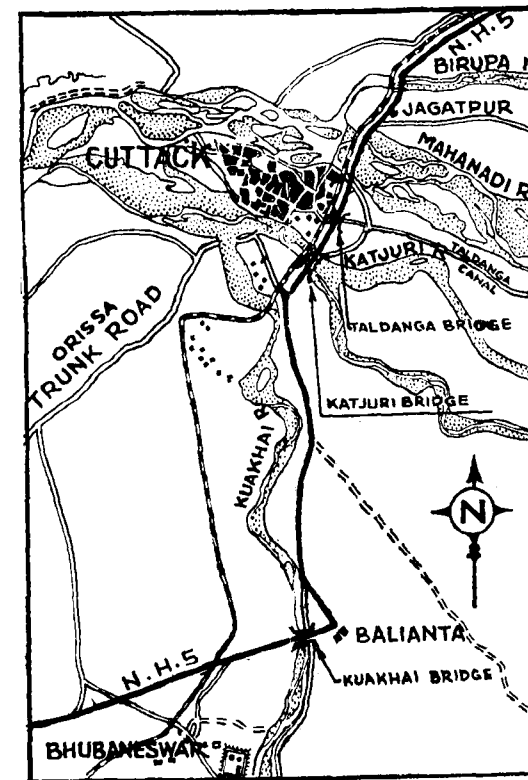
Out of these, alternative No. 3 was considered the best on technical grounds, but some changes were considered to be necessary. The design which emerged finally consisted of—

- (1) Main R. C. solid slab of varying thickness of 103 ft span between bearings;
- (2) Two cantilever spans of 47 ft each of R. C. solid slab of varying thickness;
- (3) Two suspended spans of 3 ft each between the cantilever spans and piers;

THE TALDANGA BRIDGE



The railway bridge upstream can be seen in the background



- (4) A semi-circular masonry arch span of 29 ft 6 in. and three segmental masonry arch spans of 20 ft each in the approach.

The main dimensions and other particulars of the bridge are:

Overall length including abutments, 317 ft; spans, as described above; shaft height of main piers, 29 ft; batter in piers, 1 in 24; carriageway, 24 ft; total width of superstructure, 26 ft 6 in.; thickness of slab at bearings, 7 ft; at the centre, 2 ft 9 in.; at the ends of cantilevers, 2 ft 11 in.

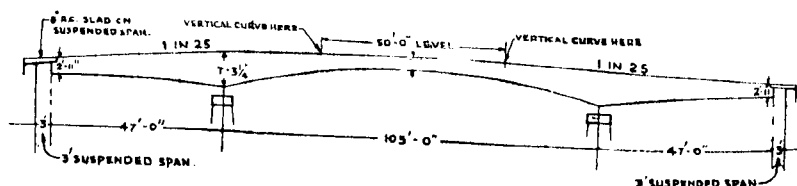
Live load, Indian Roads Congress Class AA on the Carriageway.

Foundations: open, on sandy soil; average pressure intensity on the foundation, 1.5 tons per sq. ft.

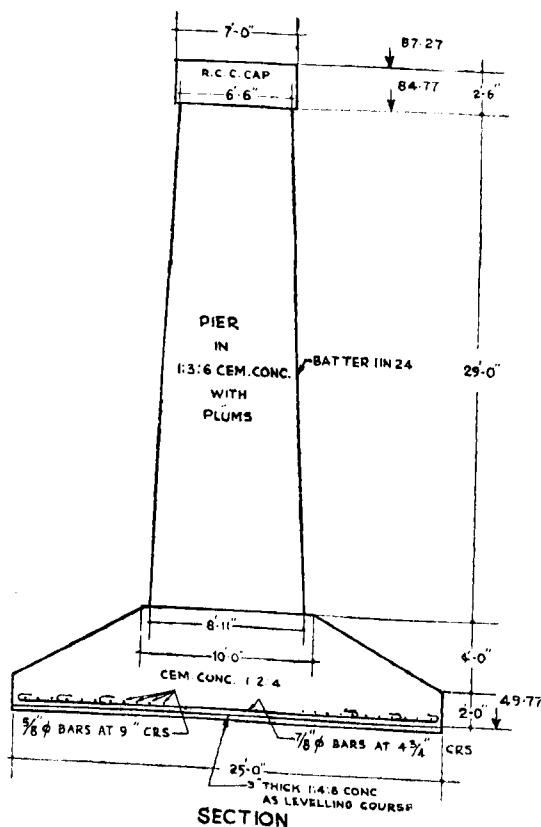
Discharge in the canal, 1100 cusecs; velocity, 3 ft per second.

Costs

The total cost of the bridge was Rs 365,000. As the quotation was on a lump sum basis the break-up of this amount is given only approximately:



Cost per running foot, Rs 1152; per square foot of carriage-way, Rs 48; per square foot of entire elevation, Rs 41; and per cubic foot of bridge, Rs 1/9/-. The approximate cost per running foot of the concrete portion would appear to be Rs 1360/- and of the masonry portion Rs 800/-.



Special Features

The original designs provided for brick-work foundations and piers. As good bricks were not available in the locality, the two main piers of the concrete spans were made of 1:3:6 cement concrete with plums on 1:2:4 reinforced concrete bases. The other piers and abutments were built of brick-work in cement. All piers were made with semi-circular ends.

A length of 50 ft at the middle of the main span was kept level. From here the formation sloped away in both directions at 1 in 25 so as to keep down the height of the embankment. The

height of the bridge at the centre of the main span was determined by considerations of the minimum clearance (25 ft) specified for navigation.

Vertical curves have been provided at all changes of gradient on the bridge. This was found particularly necessary as the wearing surface is of cement concrete. The vertical curves have been designed for a vehicle speed of 25 m. p. h.

The 3-ft suspended spans were introduced at a late stage because calculations showed that the end of a cantilever span would sag by about 4 inches under a design load and unless some suitable provisions were made a vehicle passing over the bridge at such a time would bump against the edge of the abutment wall. The suspended spans provided a means of adjustment.

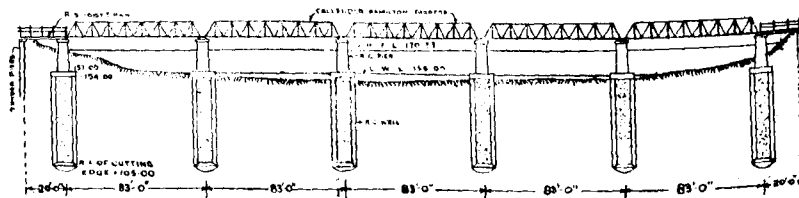
Tests

The bridge has been tested twice. The first test demonstrated amply that the structure was strong enough to withstand the load for which it was designed. The test did not, however, provide satisfactory scientific data as the bridge structure was not allowed to regain its original form, as far as possible, after the removal of each test load and before imposing another load system. As this was the longest solid slab span in the country it was considered desirable to test it again more systematically so as to obtain data for guidance in future designs.

9. THE KALJANI, THE GADADHAR AND THE RAIDAK BRIDGES

As a result of the partition of India in 1947 communications between Assam and the rest of the Indian Union were severed. No means of communication—railway, road or navigable river—was available which did not, at one point or other, pass through Pakistan. To overcome this handicap and to connect Assam with the rest of India, the Government of India undertook the Assam Rail Link project and the construction of the Assam Access Road. Work on these projects started before the end of 1947.

The Cooch Behar and Assam sections of the Assam Access Road project involved the construction or re-construction of several minor bridges and culverts, the strengthening of some bridges and the setting up efficient ferry arrangements on the Kaljani, the Gadadhar, and the Raidak rivers as a temporary expedient.



The alignment of the Cooch Behar section of the Highway was finalized in 1949 when investigation of sites for permanent bridges over these rivers was undertaken.

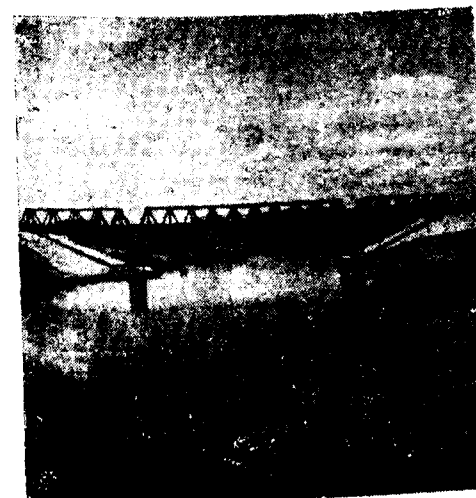
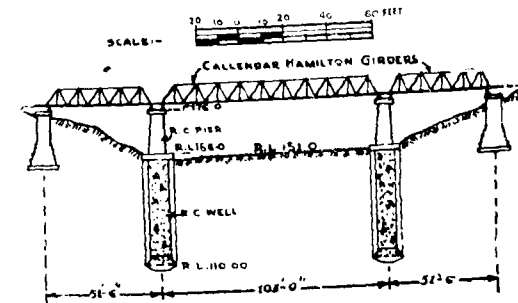
The three rivers rise from the same series of hills in Bhutan and flow through similar country. The catchment consisting of forest hills, tea gardens, and cultivated plains, is also similar in all the three cases. The foundation problems were identical and were solved in the same manner on the three rivers. It is therefore, proposed to deal with all the three bridges in this article.

The rivers Kaljani, Gadadhar and Raidak are perennial streams and in dry weather carry about 6 ft of water flowing at a velocity of about 2 ft per second.

During floods the water level rises by about 14 ft and the velocity to about 6 ft per second, accompanied by appreciable scour. The Kaljani and the Gadadhar also carry brushwood. Piers consisting of timber piles, which it would not be possible to drive more than 25 ft into the river bed or brace properly, were not considered advisable. Screwerete piles, too, were considered

but equipment to drive these piles was not easily available. It was, therefore, decided to go ahead with the construction of permanent masonry substructures with Callender-Hamilton trusses as superstructure, as a temporary arrangement pending the construction of permanent superstructures when traffic developed. The piers have been built for 2-lane bridges but the temporary Callender-Hamilton superstructures provide only one-lane accommodation.

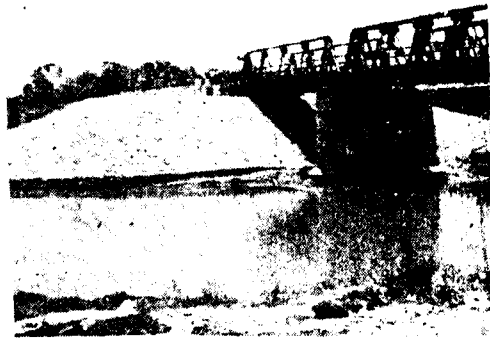
At the bridge sites the rivers are out of their boulder stages and their courses are very tortuous and unstable. The bridges were sited at as straight reaches as could be found in the circumstances and provision was made for training works.



The Gadadhar Bridge—Completed.

Photos in this article by courtesy of Shri R.P.Chanda, Ex. Engr., Cooch Behar (W. B.)

The substratum was found to be silt and sand, with some shingle at 30 ft or more below the bed level. The foundations consist of twin (figure-of-8) wells of lightly reinforced 1:3:6 cement concrete steining on heavily reinforced concrete well curbs, with mild steel cutting edges. The cutting edges consist of 9 in. $\times$ $\frac{3}{4}$ in. mild steel plates welded to 3 in. $\times$ 3 in. $\times$ $\frac{1}{2}$ in. mild steel angles. The well caps consist of 3 ft thick doubly reinforced concrete slabs.



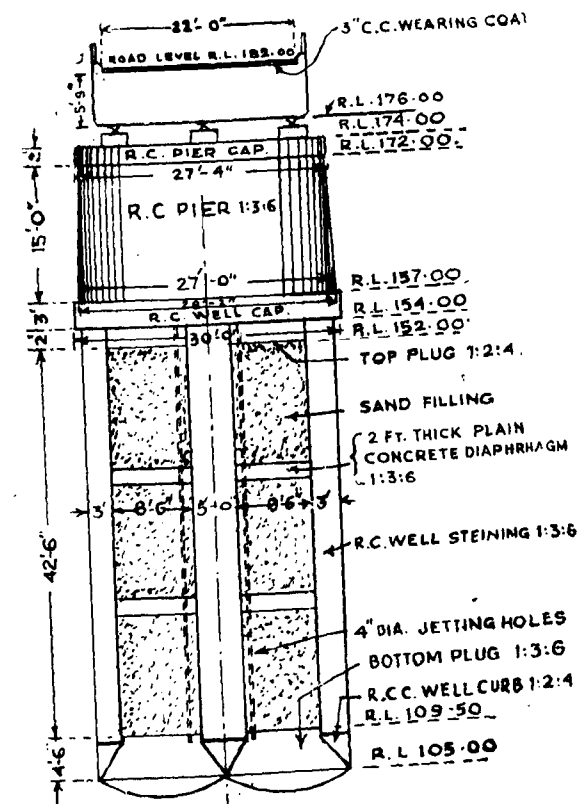
Gadadhar Bridge East Abutment—showing scour against the toewall of bank.

The piers are of lightly reinforced mass concrete (1:3:6) with semi-circular noses. The top dimensions of the piers, exclusive of the semi-circular ends, are 21 ft $\times$ 6 ft. The pier caps are reinforced concrete slabs, 2 ft thick on 83 ft spans and 2½ ft thick on the 103 ft span.

The foundations and piers are designed for a reinforced concrete balanced cantilever and suspended span type of bridge carrying 2-lanes of I.R.C. Class A or a single lane of I.R.C. Class AA loading.

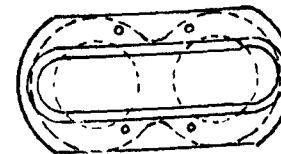
The span lengths were dictated by considerations of the material immediately available for the superstructure. This happened, for the most part, to be 80 ft span Callendar-Hamilton trusses. The piers were, therefore, spaced 83 ft apart centre to centre. The Kaljani and the Raidak bridges were to have five

spans of 83 ft each and two shore spans of 41 ft 6 inches. The Gadadhar bridge was designed for a central span of 103 ft and two side spans of 51 ft 6 inches.



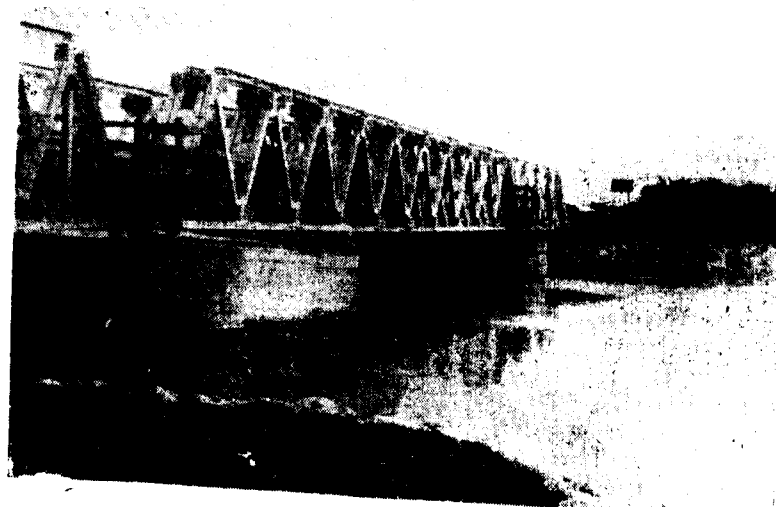
NOTE:-

R.L.'S REFER
TO KALJANI
BRIDGE ONLY.





Raidak Bridge—Showing a record flood (on 12-13, July 1952)



Gadadhar Bridge—under a high flood just falling down. Bank slopes pitched with stone boulders, stood well under floods,

The main features of the bridges are tabulated below :

	Kaljani & Raidak Bridges	Gadadhar Bridge
1. Spans	5 × 83 ft 2 × 41½ ft	1 × 103 ft 2 × 51½ ft
2. Length between abutments 6	492 (as actually built 455 ft)	200 ft
3. Length of well shaft	52 ft	46 ft
4. Height of pier shaft	15 ft	17½ ft
5. Catchment area	Kaljani 433 sq. miles. Raidak 1200 sq. miles.	114 sq. miles.
6. Maximum bearing pressure on foundation soil	6 tons per sq. ft.	6 tons per sq. ft.
7. Minimum clearance above H.F.L. at centre of span	8 ft	8 ft
8. Maximum velocity of currents to be allowed for in design	10 ft per second	10½ ft per second

Costs

The cost of the bridges per running foot, as constructed at present with Callender-Hamilton superstructures was about Rs 2,000/- for the Gadadhar and the Raidak bridges and Rs 2,130/- for the Kaljani bridge. These unit costs do not include the cost of training works.



The Raidak Bridge—Callender-Hamilton truss assembled on the bank before launching.

Well sinking on these bridges has been the most costly item. It cost Rs 1,100/- per running foot of the well shaft from the cutting edge to the top of the well cap, even where the well cap

was actually above the ground.

No Snags

There have been no snags in the construction work. Except for some minor tilts and shifts in the Kaljani foundation wells, the work has proceeded smoothly. The design of the permanent superstructure on that bridge will be suitably modified to allow for the effects of these displacements of the foundations.



A safety platform on the Kaljani Bridge.

One serious development has taken place on the Kaljani bridge. This bridge was sited normal to the current but in the last two rainy seasons the river is said to have shifted its course and the bridge has now a skew of 40 degrees. Attempts are being made to train the river into its original course.



Raidak Bridge during construction.

As sufficient Callender-Hamilton truss material could not be obtained for the four $41\frac{1}{2}$ - ft shore spans on the Kaljani and the Raidak bridges, these spans were reduced to 20 feet. The abutment piers were made of timber piles instead of cement concrete and bridged with R. S. joists.

10. THE POONPOON BRIDGE

The Poonpoo river crosses the Grand Trunk Road (N.H. 2) in the Gaya district of Bihar at about 331 miles from Calcutta. Prior to the construction of this bridge the G. T. Road crossed this river over a steel girder bridge on stone masonry piers and abutments. The old bridge had three spans of 60 ft each and was suitable for a live load of 8 tons only.

During World War II when the north-eastern frontier of India was threatened, the Government of India authorized the Bihar Government to undertake as quickly as possible the work of raising the G. T. Road in Bihar to Class 12 standard. This programme included the construction of a new bridge over the Poonpoo river. Later, the newly proposed Poonpoo bridge, like other bridges on National Highways, was designed to take I.R.C. AA loading.

The war threat subsided before any progress was made on constructing a new bridge, but the damage caused to the old bridge by a flood in 1946 lent urgency to the project. A pier on the Calcutta side was washed away and the other component parts of the bridge were also damaged to some extent. With a view to maintain light traffic without incurring heavy expenditure the old bridge was repaired and its steel girders set up on screw piles.

The old bridge had a waterway of 187 ft and an average depth of flow of about 20 ft during high floods. Calculations showed that it could be expected to pass a discharge of only 27,000 cusecs under such conditions, while the discharge of the river according to recognised run-off formulae was about thrice this figure. In point of fact the road on both sides of the old bridge was frequently overtopped in high floods by as much as 2 to 4 feet, and occasionally the banks were breached for fairly long periods thus causing great obstruction to traffic.

It was, therefore, decided in the new design to increase the waterway and four spans of 70 ft clear were provided on the new bridge. Overtopping of the road hereafter, if it occurs at all, is not likely to be serious.

Site

The new bridge is located with its centre line 65 ft above that of the old bridge and parallel to it. As the old bridge was seriously obstructing the flow in the river it was dismantled as soon as the new bridge was completed in 1948.

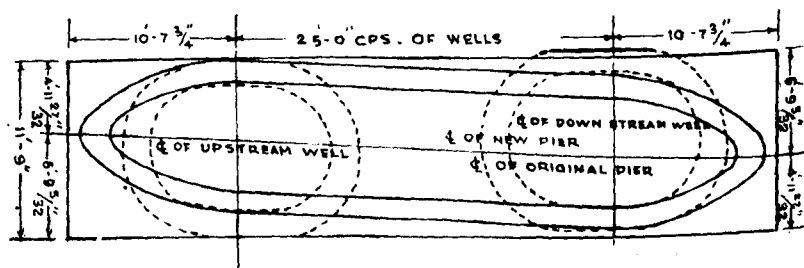
Costs

The work was executed on a lump sum contract for Rs 3,67,419 for the substructure, and Rs 2,63,117 for the superstructure. These figures did not include the cost of bricks (Rs 63,641), the wages of divers, signalmen, etc., employed on well sinking (Rs 20,291), the cost of approaches (Rs 88,000), the cost of building a Shiva Temple and a Bathing Ghat on the Poonpoon in lieu of those dismantled in siting the new bridge (Rs 6,878) the cost of dismantling the old bridge (Rs 15,013), the departmental charges (Rs 87,780) and so on. The total cost is expected to be about Rs 9,67,000. The cost per running foot of the bridge works out to about Rs 2,700, and per square foot of carriageway, Rs 123.

The rate for extra work of well sinking (bricks supplied free) below the contract levels was Rs 189/- per running foot and the rebate for founding wells above the contract levels, Rs 161/- per running foot.

Special Features

The wells of the central pier gave some trouble. While the upstream well sank true, the downstream one shifted out of the correct position towards the west by 1 ft 5 in. and was out of plumb leaning towards the east by $3\frac{1}{2}$ inches. This meant that the lateral departure of the base of the well from the true centre line was 1 ft 8 $\frac{1}{2}$ in., and of the top of the well 1 ft 5 inches. It is understood that the well struck a buried capping stone of an old pier on one side and tilted towards the other. Apparently this resulted in a slight lateral movement also. The departure from the true position was originally as much as 2 ft 7 $\frac{1}{2}$ in. at the top with a corresponding non-verticality. By tying the well with taut ropes and judicious digging and sinking the displacement was reduced to 1 ft 8 $\frac{1}{2}$ inches.



The pier was built eccentrically over both the wells, thus reducing the eccentricity by half. As a result one span became 73 ft 8 in. between bearings and the adjacent span 74 ft 4 inches.

The bridge was thrown open to traffic in April, 1949.

THE POONPOON BRIDGE



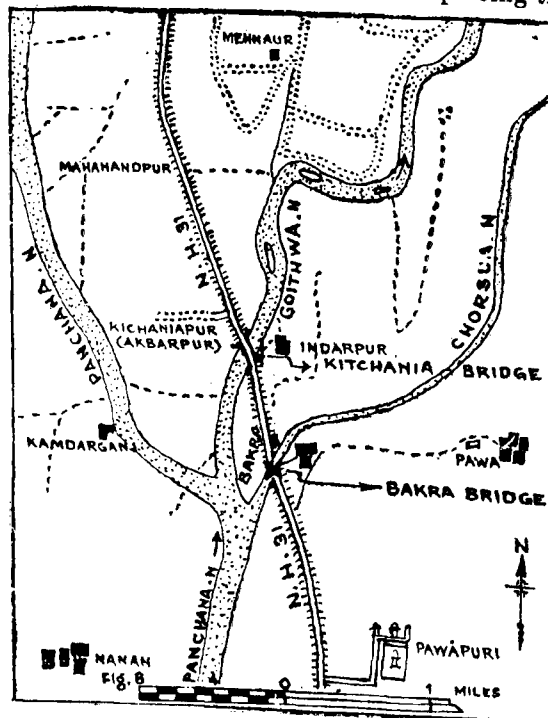
Another view of the Poonpoon bridge.
(By courtesy of M/S Kumardhubi Engineering Works Ltd., Kumardhubi.)

11. THE BAKHRA AND KITCHINIA BRIDGES

BAKHRA and Kitchinia are two small villages within a mile of each other on the Patna-Ranchi Road in Bihar. A portion of this road, namely that from Barhi to Bakhtiarpur, falls along the accepted alignment of N. H. 31. In this portion the National Highway crosses the river Panchana which takes off from the hills near the famous mica mines of Koderma. The Panchana is a flashy torrent which remains practically dry throughout the year and carries water only during the rains. About half a mile above the road-crossing, the river divides into three branches—the Chorsua, the Goithwa, and the Panchana itself. From this point onwards the first two branches are more active than the main river Panchana which is practically dead and silted up.

The Patna-Ranchi Road used to cross the two main branches by means of masonry bridges with light steel girders, designed for a live load of about 4 tons only. The roadway on these bridges was less than 10 ft wide.

The bridge over the Chorsua (Bakhra) branch was washed away in August 1948 on account of growth of discharge in this branch. It was decided that while replacing the Bakhra bridge that over the Kitchinia should also be replaced, for the reasons to be explained below, by one designed to National Highway standards.



Map of locality

Since the main river divides itself into three branches and the discharge through each branch has been varying from year to year, it is not possible to assess with certainty the discharge that will pass down each branch in the years to come.

The selected waterways for the bridges were fixed from a study of the cross-sections of the river and their behaviours

KITCHINIA BRIDGE



A view of the Completed Bridge.

[By Courtesy of M/s. J. C. Gammon Ltd., Bombay.]

loading, a stream velocity of 10 ft per sec., a wind pressure of 20 lb per sq. ft. a temperature variation of ± 30 deg. F. and a safe bearing capacity of 8 tons per sq. ft. (7½ tons on the Bakhra bridge) in good yellow earth strata.

Each pier consists of two circular columns, well braced at the top and bottom, by pier and well caps. In the two intermediate piers on the Kitchinia bridge diaphragm walls are provided between the columns (no such diaphragm walls are provided on the Bakhra bridge).

Bakhra Bridge

The Bakhra bridge is similar to the Kitchinia bridge in general detail. It is 227 ft between abutment faces, and has a lineal waterway of 216 ft 8 inches. The spans are 68.5 ft, 90 ft, and 68.5 ft. The suspended beams and the end beams over the abutments rest on sliding plates. On the piers the beams rest on lead sheets.

Cost

The break-up of the cost is tabulated below:

	Kitchinia bridge (Rupees)	Bakhra bridge (Rupees)
Foundations	2,68,500	1,79,000
Piers	34,000	20,000
Abutments and wing walls	54,000	65,000
Superstructure	2,08,500	1,16,000
Total	5,65,000	3,80,000
Cost per running foot	1,210	1,390
Cost per sq. ft. of Carriageway	50	58
Cost per sq. ft. of entire elevation	17.5	20.3
Cost per cu. ft. of Bridge	0.73	0.85

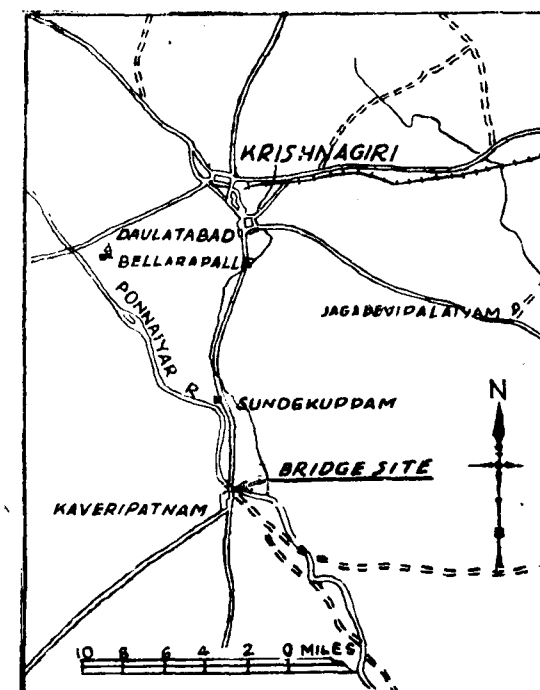
The bridges were opened to traffic in 1951, having taken about a year to build. The contractors were Messrs. J. C. Gammon Ltd., who have kindly supplied the photos, drawings, and cost data.

12. THE SOUTH PENNAR BRIDGE AT KAVERIPATNAM

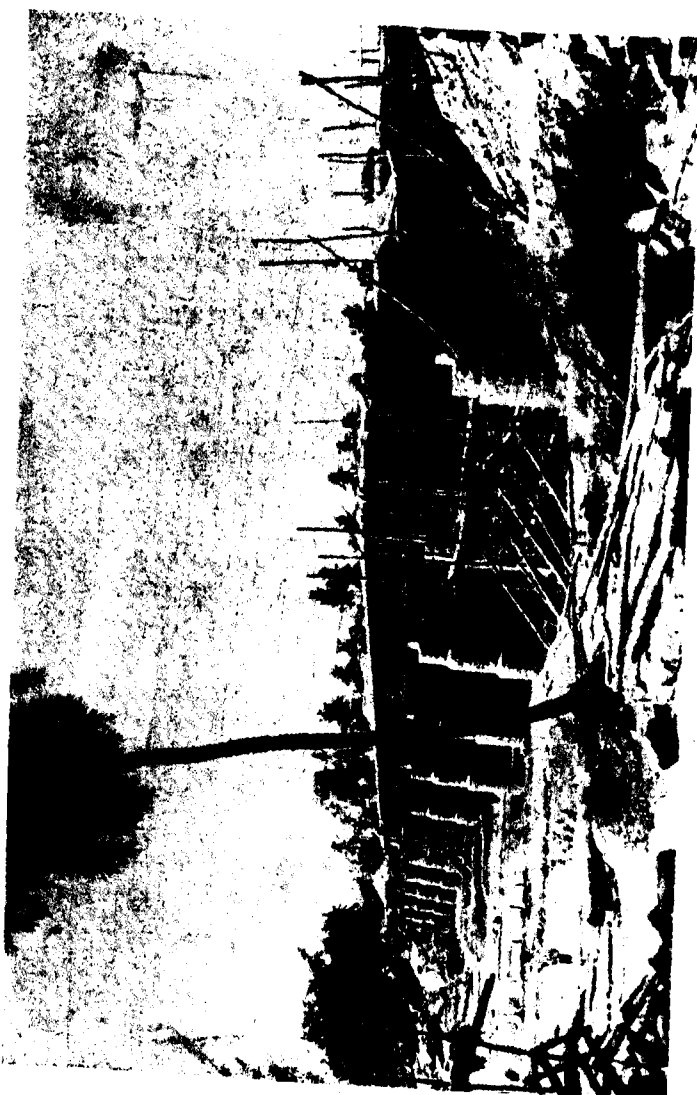
When the alignment of N. H. 7, the Banaras-Cape Comorin road, was decided by the Government of India, a section of the Madras-Calicut road was adopted as a part of that Highway. About 166 miles from Madras this road crosses the South Pennar river. The bridge at this point, built in 1913, was a structure of twelve spans, each 42 ft 6 in. clear. The superstructure consisted of two 30 in. x 8 in. built-up plate girders, 10 ft 3 in. apart, with 7 in x 7 in. rolled steel joist cross girders at 3 ft centres. Brickwork arches spanning the spaces between the cross girders with 9 in. metalling formed the deck. The width of the carriageway was only 9 ft between kerbs, and there were no footpaths.

As the bridge was near populous villages there was considerable vehicular and pedestrian traffic over it. The pedestrian traffic was particularly heavy on "shandy" days.

["Shandy" is a local term meaning 'weekly market', and has no connection with drinks or drinking.]



THE SOUTH PENNAR BRIDGE AT KAVERIPATNAM.



[Photos by Courtesy of the Chief Engineer (Highways), : Madras]
The Bridge under Construction

THE SOUTH PENNAR BRIDGE

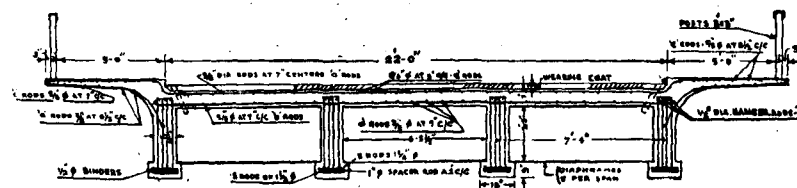
The nearest railway is 34 miles away and consequently passenger and goods traffic is very heavy over this section of the road. The superstructure of the girder bridge was so flimsy that vibration under traffic could be easily seen and felt. The old bridge was thus unsuitable for the requirements both of a National Highway and of the local traffic. A stronger and a wider bridge was needed at the site than the old structure.

As the old bridge had successfully withstood all the heavy floods since its construction in 1913, it was proposed to retain the existing waterway, and merely to lengthen the piers and replace the superstructure with a wider and stronger one.

The old piers were 4 ft thick at the top and 13 ft long excluding the semi-circular cut waters. There were no abutments proper, but only two 7-ft thick abutment piers, encased within revetments at the two ends of the bridge.

The piers were founded on rock at 4 to 8 ft below the bed level. The work of extending them did not present any special problem. Work started towards the end of June 1950 and the substructure was complete by the middle of April 1951. In two piers, pockets of soft rock went down to 14 ft below bed level. These were removed and the cavities filled with cement concrete. The foundations were built in 1 : 3 : 6 cement concrete up to the level of the river bed. The piers were built in coursed rubble masonry.

To reduce inconvenience to traffic to the minimum, the old superstructure was removed only when the substructure extensions were complete. Then 18-in thick through R. C. pier caps were laid over the piers. The new superstructure consists of T-beam-cum-slab construction, simply supported over the piers so that, in the event of unequal settlement of pier extensions, the stresses in one span should not affect the adjacent spans. The details of the design are shown in the figures. The T-beams of each span have fixed bearings on one pier and sliding bearings on the other.



The Cost-plus System

The quotation of the only firm who tendered to carry out the work exceeded the official estimate of Rs 2.45 lakhs by 62.65 per cent. This was not accepted and a contract on the basis of cost plus 15 per cent was negotiated with the same firm.



[Photo : By Courtesy of the Chief Engineer (Highways), Madras.]

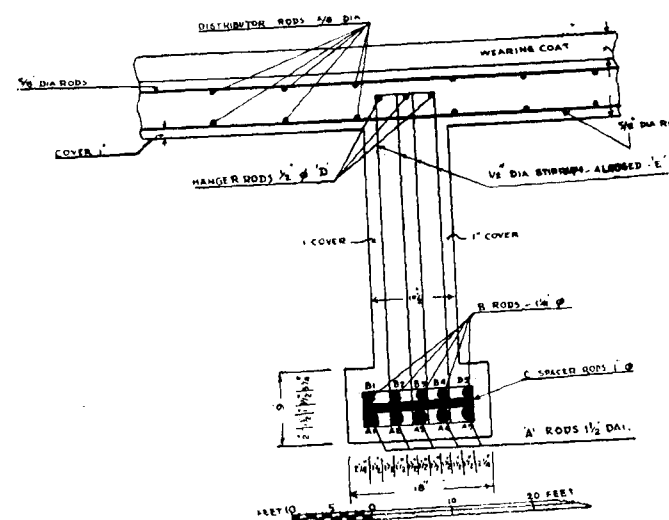
The photo shows the Superstructure in "cross sections".

The joint between the old and new stone masonry is also clearly seen.

The contract agreement defined the word 'cost' as :

- (1) the cost of all labour and materials ;
- (2) salaries of contractors' work-charged staff employed on the work but not of the supervisory staff ;
- (3) contractors' preliminary expenses for recruiting labour, sheds, water supply, medical, and sanitary arrangements ;

- (4) compensation to workmen for injuries or death ;



- (5) rental of equipment used on the work at pre-determined rates ,
- (6) cost of transporting equipment to and from the work site, insuring, and overhauling at the end of the work ;
- (7) losses on bonafide advances made by contractors to suppliers and labourers ;
- (8) cost of small tools.

Costs

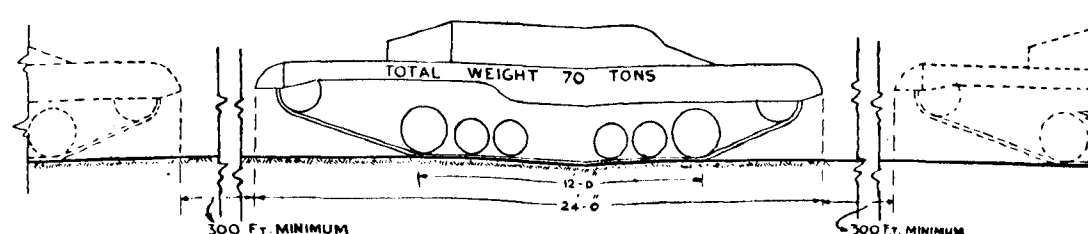
The work has cost Rs 4.02 lakhs. Analysed unit costs are not yet available. The work was completed in April 1952.

I. R. C. LIVE LOADS FOR HIGHWAY BRIDGES STANDARD VEHICLES AND TRAINS (REVISED 1948)

FIG. NO 205

PLATE I

CLASS "AA" TRACKED VEHICLE

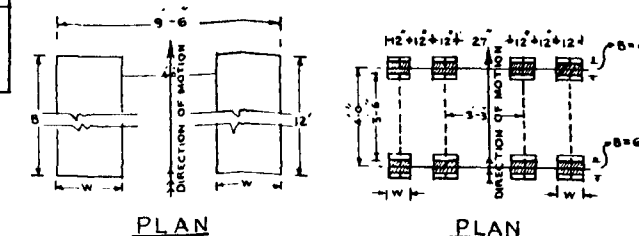
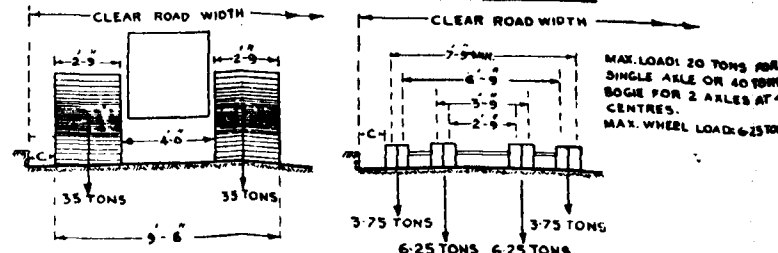


CLASS OF LOADING	PARTICULARS OF THE STANDARDS VEHICLE AND TRAINS	AXLE LOAD	GROUND CONTACT AREA
CLASS "AA"	The vehicle sketched above subject to the restriction that the nose to tail distance between two successive vehicles is not less than 300 feet. No other live load shall cover any part of the carriageway or the bridge when this train of vehicles is crossing the bridge. (a) Wheeled vehicles (b) Tracked vehicles	20 TONS (max)	INCHES 6 14.6 18 31

NOTE:-

1. The nose to tail spacing between two successive vehicles shall not be less than 300 feet.
2. No other live load shall cover any part of the carriageway of the bridge when this vehicle is crossing the bridge.

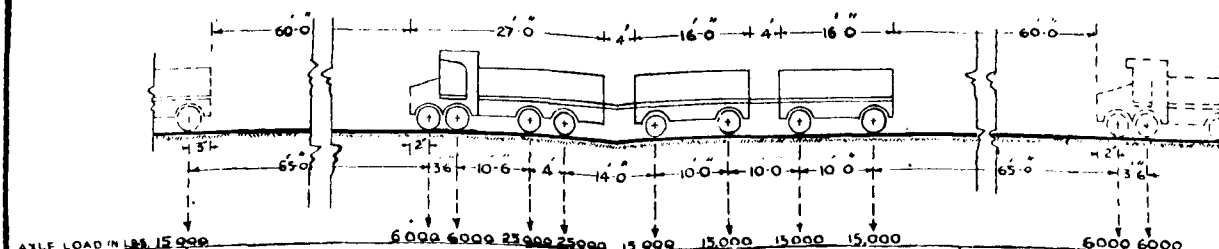
WHEELED VEHICLE



PLAN

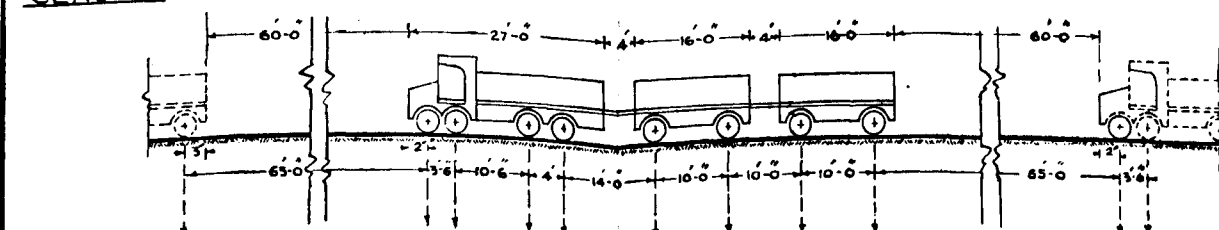
PLAN

CLASS "A"



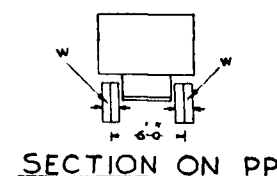
CLASS OF LOADING	PARTICULARS OF THE STANDARD VEHICLES AND TRAINS.	AXLE LOAD	GROUND CONTACT AREA
CLASS "A"	The train sketched above subject to the restriction that the minimum space (nose to tail) between two successive trains shall be 60 ft. No other live load shall cover any part of the carriageway when this train of vehicles (or trains of vehicles in case of a multilane bridge) is crossing the bridge.	25,000 LBS 18,000 LBS 8,000 LBS	INCHES 10 20 15 8

CLASS "B"

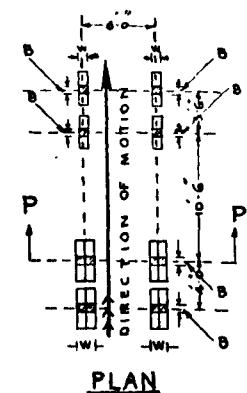


CLASS OF LOADING	PARTICULARS OF THE STANDARD VEHICLES AND TRAINS	AXLE LOAD	GROUND CONTACT AREA
CLASS "B"	The train sketched above subject to the restriction that the minimum space (nose to tail) between two successive train shall be 60 ft. No other live load shall cover any part of the carriageway when this train of vehicle (or trains of vehicles in the case of multilane bridge) is crossing the bridge.	18,000 LBS 9,000 LBS 3,000 LBS	INCHES 8 15 7

STANDARD VEHICLE CLASS "A" OR "B"



SECTION ON PP



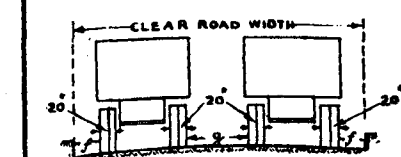
PLAN

LIMITING POSITIONS OF THE STANDARD VEHICLE OR A TRAIN OF VEHICLES WITH REFERENCE TO A TRAFFIC LANE.

CLASS "AA"

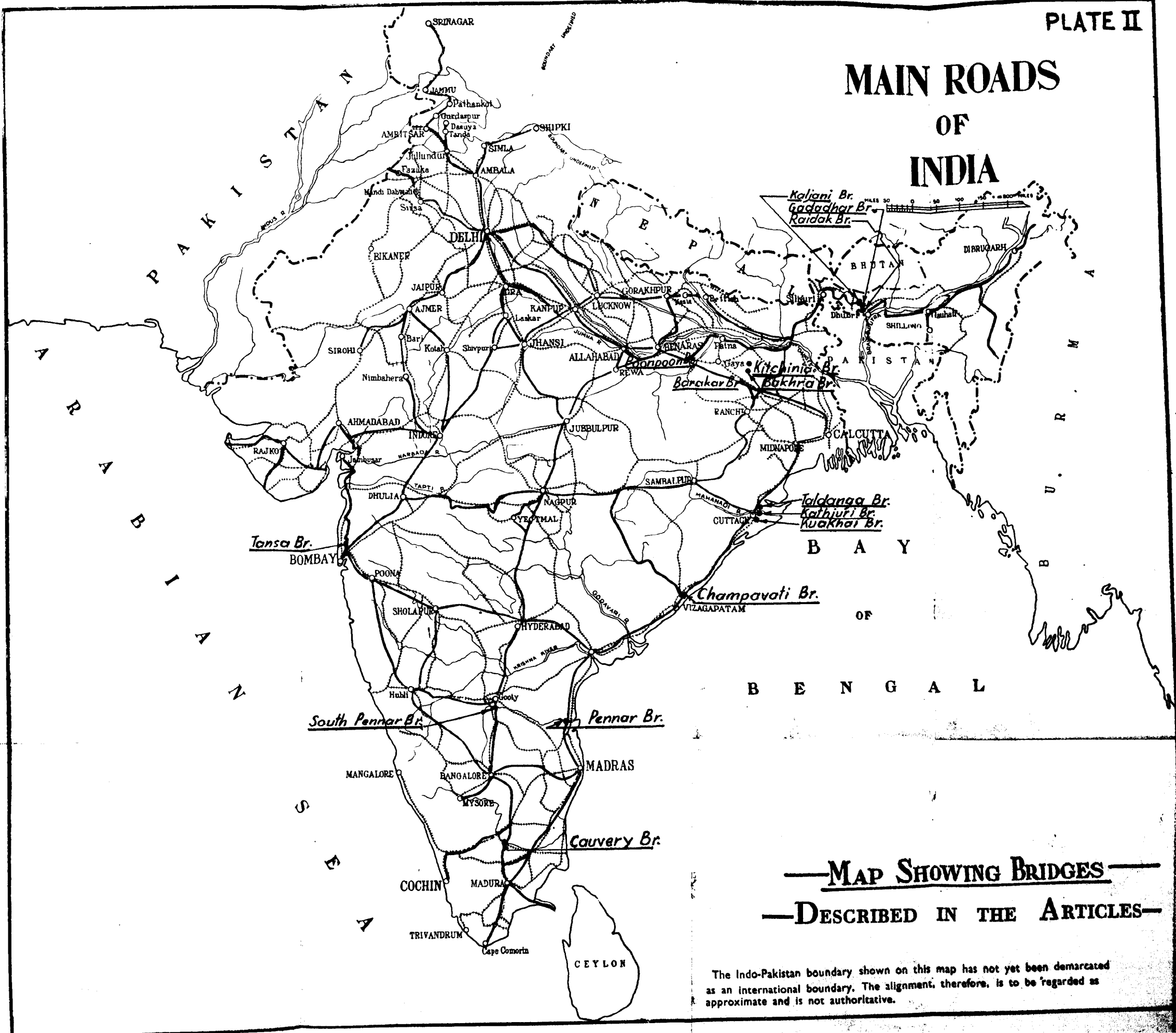
CLEAR ROAD WIDTH	MINIMUM VALUE OF C i.e. THE CLEARANCE BETWEEN THE ROADWAY FACE OF THE KERB AND THE OUTER EDGE OF THE WHEEL OR TRACK
10 OR LESS	0
10 TO 12'-6"	INCREASING UNIFORMLY FROM 0 TO 6"
ABOVE 12'-6"	1'-0"
LESS THAN 16'	2'-0"
16' OR ABOVE	4'-0"

CLASS "A"



CLEAR ROAD WIDTH	g = MINIMUM CLEAR SPACE BETWEEN THE ADJACENT EDGES OF WHEELS OF PASSING OR CROSSING VEHICLES ON A MULTILANE BRIDGE TO BE ASSUMED FOR PURPOSES OF DESIGN.	f = MINIMUM CLEARANCE BETWEEN THE ROADWAY FACE OF THE KERB AND THE OUTER EDGE OF THE WHEEL TO BE ASSUMED FOR PURPOSES OF DESIGN.
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MAIN ROADS OF INDIA



—MAP SHOWING BRIDGES—
—DESCRIBED IN THE ARTICLES—

The Indo-Pakistan boundary shown on this map has not yet been demarcated as an international boundary. The alignment, therefore, is to be regarded as approximate and is not authoritative.

INFORMATION SERVICE

CAUSES OF LANDSLIPS AND SUGGESTED REMEDIES

On the 11th and 12th June 1950 the hill slopes in and around Darjeeling were affected by disastrous landslips causing death and destruction of property. Communications, houses, and public services were destroyed in the town and in the neighbouring villages and tea gardens.

As a result Darjeeling was cut off from the outside world for many days. Electric and water supplies were stopped for a long period.

Half a century ago similar slips had occurred in September 1899.

At the request of the West Bengal Government the Geological Survey of India undertook investigations of the 1950 slips. The following extracts from the Geological Survey of India Report* on the subject are published for the information of highway engineers.

The annual rainfall in Darjeeling is 110 inches, most of it occurring between June and September. A few showers, off and on, are common all the year round. Landslips occur generally during the monsoon. At the time of the disastrous slips of 1899, 19.40 inches of rain fell within 24 hours. In 1950 there was a maximum precipitation of 17.88 inches in 24 hours. Figures for rainfall from the 10th to the 14th June 1950 are given below :

Date	10th	11th	12th	13th	14th
Rain	0.54	4.17	17.88	10.16	0.40

Fortunately rainfall of such intensity is not common, and disastrous and widespread landslips are therefore not an annual feature. Some slips are, however, reported practically every year and cause dislocation of communications.

Causes and Origin of Slips

A landslide may be defined as a sudden movement, in a downward and outward direction, caused by the influence of gravity of a portion of a sloping ground surface. A slip occurs because a portion of the sloping ground is actually sheared off from the parent body. The tendency for slipping down is counteracted by the shear strength of the material forming the slope. Thus slipping along a slope may occur when either the

* "Report on the Landslips in Darjeeling and neighbouring Hill Slopes in June 1950" by K. K. Dutta, December, 1951.

load on the slope increases or the shear resistance of the material decreases for one reason or other.

In and around the town of Darjeeling slips are found on every type of slope, whether formed of soil, talus, scree material or rock. Again, composite slips are seen, primarily involving soil or rock talus but sometimes the underlying rock too.

The shear resistance of soil and soil slopes is made up of a combination of internal friction and cohesion. In soil, especially clayey soil, internal friction is generally not more than 2 to 3 degrees. Its shear strength, therefore, depends almost entirely on cohesion. For some heights a soil slope is stable at very steep angles, but for large heights the safe angle decreases gradually and ultimately goes down to the angle of internal friction, commonly known as the angle of repose.

In other words, the height to which a soil-covered slope can stand, at angles steeper than its angle of internal friction, is limited. The exact safe height corresponding to different slopes can be found by testing the physical properties of the soil. A slope will safely stand to a great height if its inclination to the horizontal is flatter than its angle of repose.

In slopes formed of talus and rock debris, there is practically no cohesion and the stability of such slopes depends entirely on internal friction.

In slopes formed of rock, slips occur only along planes of weakness like joints, bedding planes, etc. Rock, free of such weak features, may stand even vertically to great heights.

The load on a slope increases when it is steepened, for example by erosion, or by human agency. Slopes are steepened by under cutting for making or maintaining roads, or developing flat terraces for house sites. A slope is steepened also by dumping material on it to widen a road or a house site.

Decrease in the shear resistance of the material forming a slope is caused by progressive structural changes in the material by weathering.

Percolation of water into the material also reduces its shear resistance. Factual evidence of this is offered by the fact that landslips usually follow a period of heavy rainfall when much of the precipitation has a chance of percolating into the material forming the slope. A continuous rainfall of about 18 inches preceded the landslips in the Darjeeling area on the 11-12th June, 1950.

During the rains the pore spaces and the voids in the slope material get filled up with water, thus increasing the unit weight and consequently the load on the material.

Again, water which enters the ground beneath a slope in turn involves an increase of pore water pressure and a decrease in the shear resistance of the material. Thus a soil or talus material which is ordinarily stable in a certain slope may cease to be so on account of the addition of water during a heavy storm.

The soil slips in the Darjeeling area were caused by one or all the factors mentioned here.

Most of the slips which had damaged houses were due to cutting back into the hillfaces for developing the house sites.

Mudrock Flow or Mudspate

Nearly all the major slips were found on slopes formed of talus material lying at angles steeper than 40 deg. Such talus material, when dry or well drained, is stable at an angle of 45 deg. and its stability is not necessarily impaired by occasional wet spells. Slips in such material, however, occur by seepage pressure of water percolating in as a result of a heavy rainstorm. When a slip starts, the saturated material rushes down carrying with it



Photo.1. Soil slips — showing the parabolic outline of the slipped portions.

big blocks of rock and eroding a channel of its own. This is known as a '**mud-rock flow**' or a '**mud-spate**.' In the particular cases of the slips investigated, though the angle of slope (40 deg.) appeared to be within the safe limit, the presence of clay within the talus derived by weathering tended to prevent drainage, thereby leading to an increase in the seepage pressure.

Slips in rock occur along certain definite planes. They may be bedding planes or foliation planes, joints, shears or other planes of weakness of a tectonic nature. The occurrence of slips along such planes depends on their inclination to the horizontal.

Sometimes, even when the planes of weakness have a gentler inclination than the angle of sliding friction, slips occur along them on account of a reduction of cohesion between the mineral constituents of the contact layer, and on account of the rise of the peizometric head. The cohesion may be reduced by weathering and the formation of clay with a low angle of internal friction.

Summarising, the stability of a slope depends on:

- (1) the angle of the slope ;
- (2) the material forming the slope ;
- (3) nature of slope, i.e., whether bare or covered by vegetation, and the nature of the vegetation ;
- (4) ground-water conditions ; and
- (5) earthquakes.

Bad drainage is an important cause of slips. Good drainage can lower the ground water level and reduce the peizometric head or the pore water pressure and thereby increase stability, whereas bad drainage has just the opposite effect. Unlined drains and storm-water channels help to recharge the ground water. Clogged drains, commonly found in 'bustec' areas, also have a similar effect.

Addition to ground water is also helped by cultivation on hill slopes. "**Terrace**" cultivation, often recommended for reducing erosion, is not equally beneficial for the stability of slopes.

Forests are given an undue importance in the protection of slopes. Tree roots get into the crevices or joints in rock and tend to loosen it. In addition, they make the slope surface porous. On the other hand, small, dense undergrowth and thick turf protect slopes both against slips and erosion. In several places in Darjeeling it was seen that while unprotected soil slopes could not

ordinarily stand to heights of more than 20 feet, turfed slopes safely stood to heights of 50 feet and sometimes even more. Where the ruling angle of slope was 40 deg. turfed slopes stood at steeper angles, up to 50 deg. to considerable heights.



Photo 2 : Another slip.

Creep.

Creep may be described as the movement, at an imperceptible rate, of the material forming a slope. Slumping along roads, which is recorded at long intervals, may be classed in this category. The average rate of creep at many places is known to be less than one foot in ten years. A very good example of creep is found in the slumping of the Cart Road to Darjeeling near milestone 23.

In the case of a slip the stresses on the slope exceed the shear strength of the material, but creep begins with a much smaller stress. This is called the **fundamental stress** of the material. When the stresses on the slope exceed this value, the slope "creeps", and if they become equal to the stress required to produce shear failure, a slip occurs. Under certain circumstances creep may develop into a slide, and a slide may be followed by creep in the slipped material. This was the case at mile 23 of the Cart Road, where slumping or creep was going on in the debris of an earlier slip.

Preventive Measures

Preventive measures are determined by the causes tending to produce a slip. These measures are :

- (i) Reducing the angle of the slope ;
- (ii) Preventing seepage ;
- (iii) Drainage ;
- (iv) Building retaining walls.

Theoretically, the stability of a slope can be increased by reducing the angle of the slope. Actually, however, this is not a practical proposition on natural slopes. It is possible in only small parts of a slope and then, too, at the cost of some other part where the slope angle may correspondingly increase.

Prevention of Percolation and Seepage

The stability of a slope can be increased by preventing percolation by covering the portion of the slope to be protected by a layer of impervious material like clay. This will, however, be practicable for small areas only. Turfing, on the other hand, should be possible on every bare soil slope. Turfed slopes remain stable up to a greater height than bare slopes owing to the fact that clayey soil adhering to the grass roots ensures an impervious blanket on the slope.

Rocky slopes with open joints can be made impervious by grouting with impervious material. Grouting should be tried where the expense can be justified.

Afforestation is also beneficial as the secretion from the tree roots helps the formation of clayey soil. As explained previously small shrubby undergrowth is preferable to big trees in this respect.

Seepage is due not only to direct rainfall on the slope under consideration, but also comes from adjoining ravines which carry the run-off. To protect the slopes near such channels, the latter should be lined.

Drainage

Pore water pressure can be lessened by preventing seepage. It can be controlled and kept within safe limits by draining out the water which might have already seeped in. Slopes formed of clayey soil cannot be drained easily but those formed of talus can be drained. In the latter case, installation of deep drains can ensure protection against slips.

Catchwater Drains

For protecting a small slope one drain along the upper boundary of the slope is generally sufficient. But for long and steep slopes, several drains at different levels may be necessary to ensure full protection. The drains should lead into natural gullies which should be lined if necessary.

Adits driven into the hillface may also be tried. In the case of rock, drainage by drill holes may be found cheaper than drains and adits. The drill holes should be horizontal and more than 6 in. in diameter.

Retaining Walls

On slopes formed of impermeable clayey soils which cannot be easily drained, retaining or revetment walls are necessary for protection against slips. But they are costly and should be built only when such expenditure is justifiable.

In Darjeeling retaining walls are built, in most cases, of dry rubble masonry according to certain standard designs. Since many of these retaining walls, including some built of reinforced concrete, failed it would appear that the standard designs adopted need examination. In designing a wall the pressure exerted against it should be computed carefully.

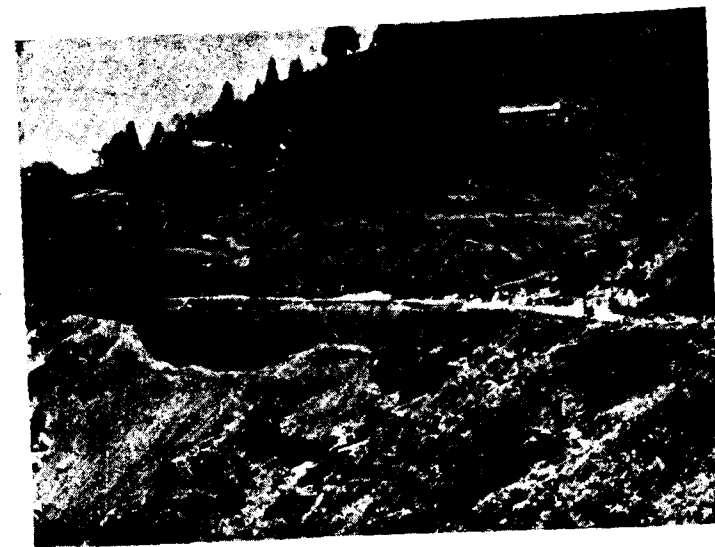


Photo 3. A road slip leading to the construction of a costly stone retaining wall.

Experience has shown that most of the retaining wall failures were caused by unsuitable foundations. It may, therefore, be useful to consult an experienced soil-technician. To be able to give proper advice, he may have to collect samples of soil and test them in a soil-mechanics laboratory.

The failure of some of the old stone revetment walls, and also probably some reinforced concrete walls, is due to seepage pressure. While the effect of seepage is usually taken into account in designing revetment walls, it cannot be completely estimated in advance. Seepage causes a seasonal variation of earth pressure behind retaining walls, sometimes amounting to nearly 30 per cent of the average value. It is, therefore, necessary to prevent the accumulation of water behind a retaining wall.

Weep Holes

Relief is afforded by providing weep holes through the retaining walls. It may be mentioned here that the reinforced concrete revetment walls in Darjeeling were not provided with weep holes. The weep holes in the stone walls were found to be choked with fine-grained material which had got in from the back. For all practical purposes, these holes were non-existent as they were not functioning at all. Again, these weep holes were merely in the form of gaps in the courses of masonry. This practice allows seepage through the joints in the masonry into the foundation at the toe of the structure, and weakens the foundation. A proper weep hole can be provided by embedding a 4 in. pipe in the masonry. Rows of weep holes should be spaced 5 ft apart vertically and the holes should be the same distance apart horizontally. About a cubic foot of crushed stone should surround the inner end of each weephole pipe to prevent its being choked up. A layer of crushed stone, about a foot thick, behind the entire retaining walls, with a row of holes at the bottom of the wall, is expected to prove useful in draining the backfill. The drainage from the weep holes should be carried off in a lined channel in the front of the wall.



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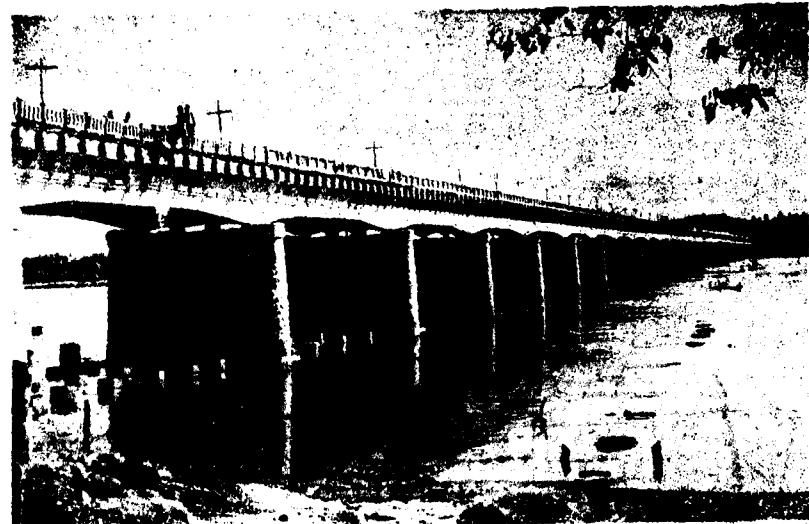
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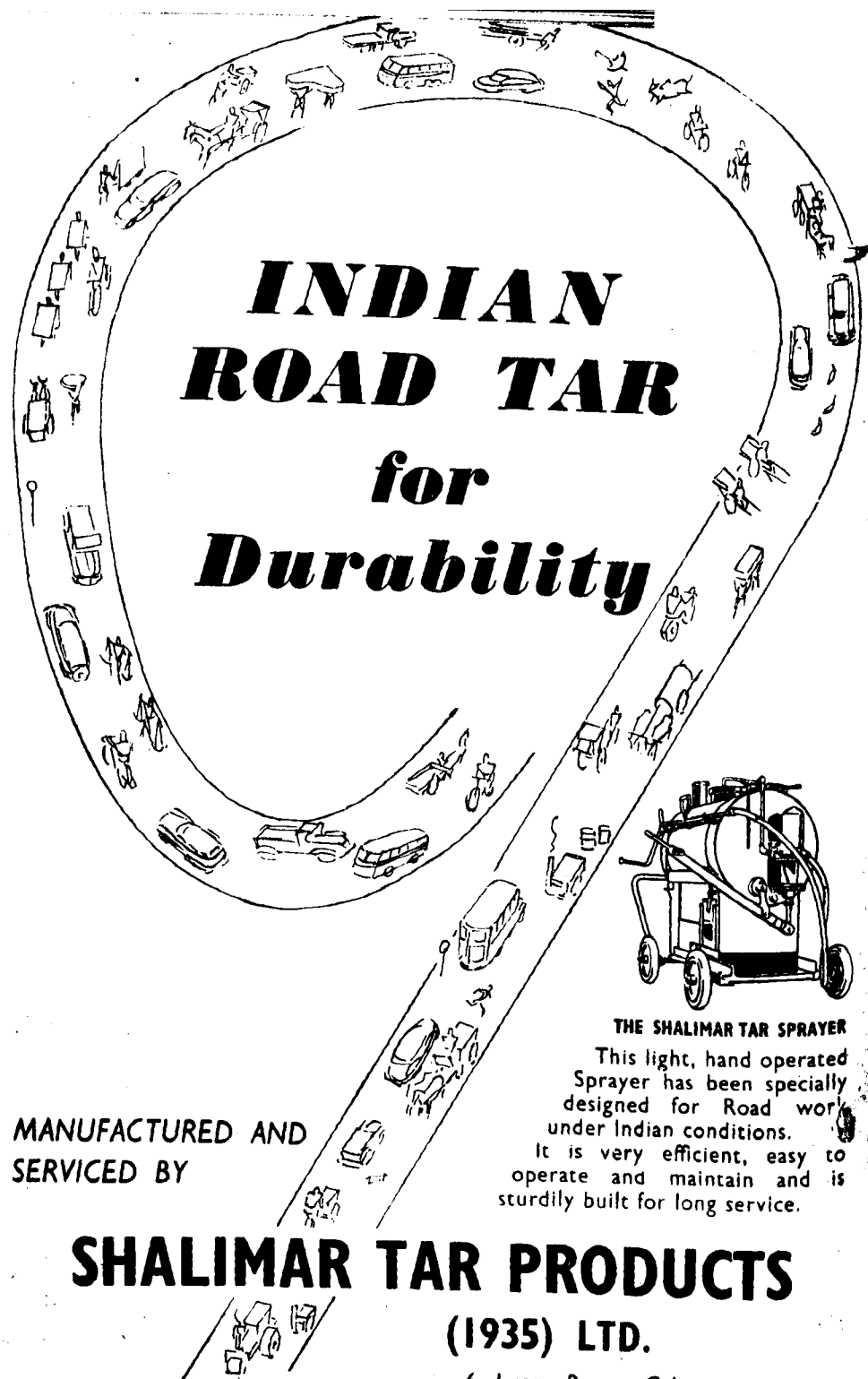


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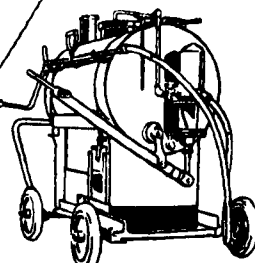


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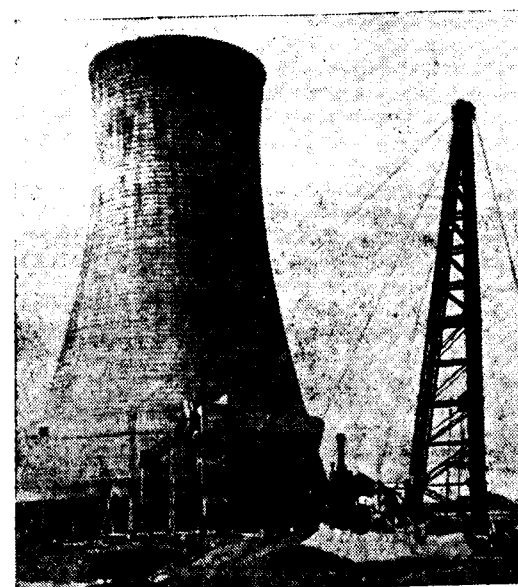
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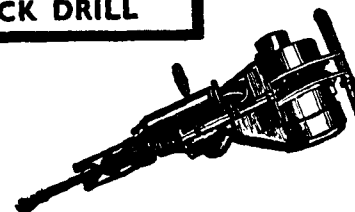
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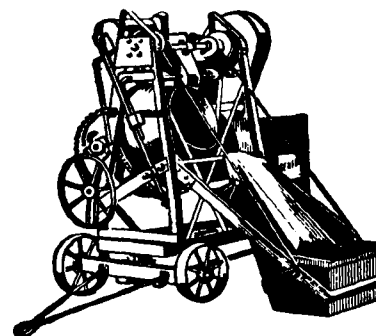
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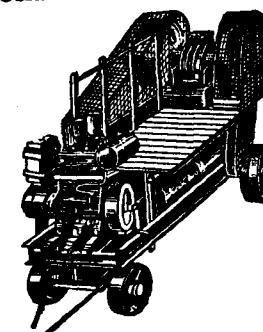


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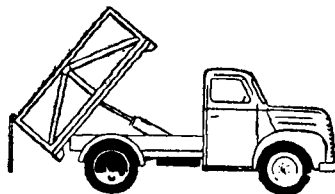
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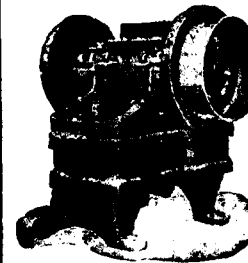
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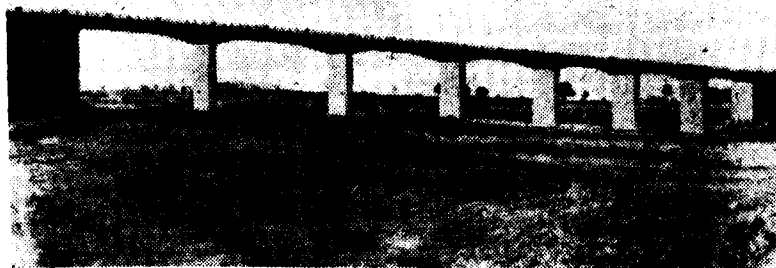
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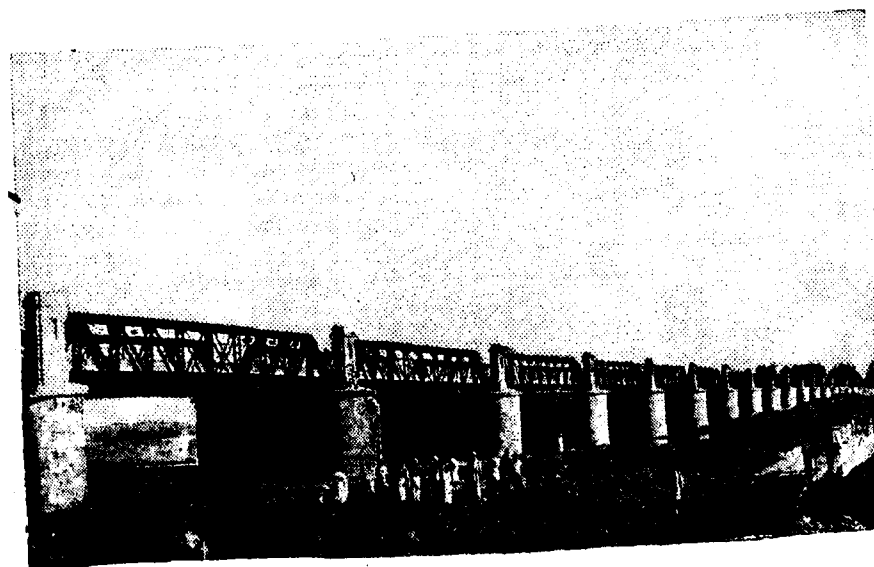
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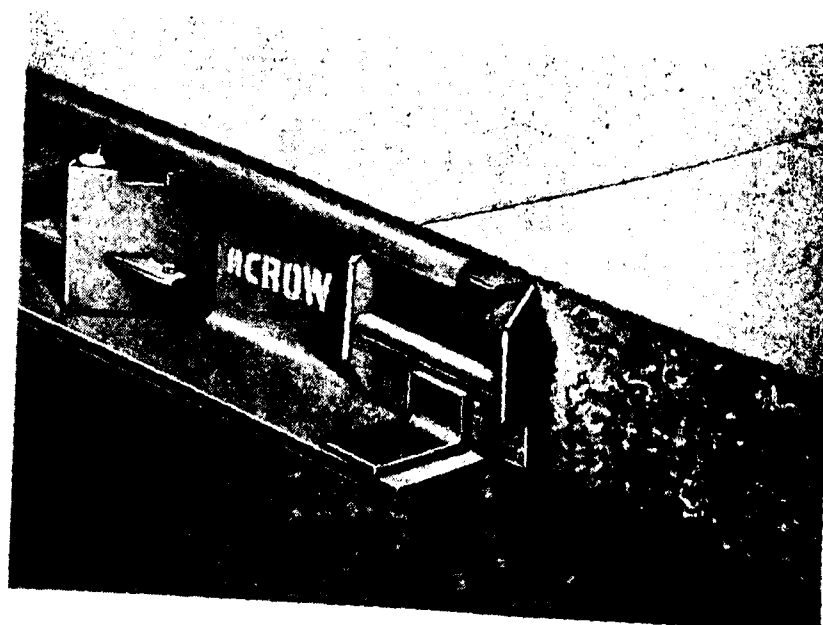
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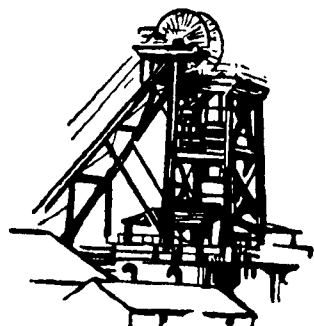
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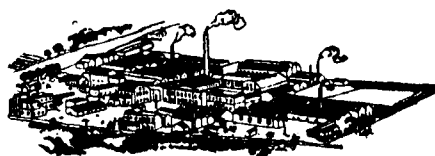
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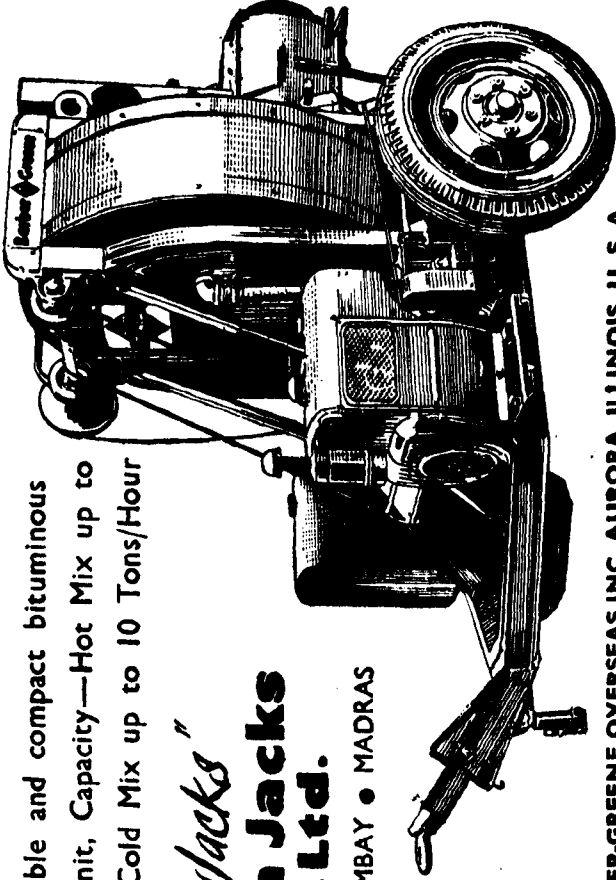
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