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**"DESIGN AND CONSTRUCTION OF THE
PRESTRESSED CONCRETE BRIDGE
ACROSS RIVER COLEROON"**

By

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CONTENTS

	Page
1. Introduction	150
2. The Site	151
3. Hydraulic Particulars	151
4. Selection of the Span	152
5. Soil Profile at the Site	152
6. Foundations	152
7. Installation of Scour Meters	156
8. River Training	156
9. Tests on Foundation Soils	157
10. Interpretation of Load and Soil Tests	169

THE SUPERSTRUCTURE

11. Some Features of Design	179
12. Materials	190
13. Production Yard	193
14. Production of Beams	197
15. Calculations of Stresses and Elongations in Cables	207
16. Prestressing Procedure and Problems	217
17. Launching of Beams	231

Appendices

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SYNOPSIS

The bridge across Coleroon river on the Chidambaram-Sirkali road consists of 14 spans of 150 ft each. The superstructure of the bridge consists of unbalanced cantilever prestressed precast T-beams. The deck of the bridge is cast at site and is transversely prestressed. The system of prestressing adopted is the Freyssinet type.

The foundations consist of 3 square wells under each pier. Originally they were to be taken down 5 to 6 ft into compact clay. Actually, the wells have been sunk to depths varying from 50 to 55 ft below the bed level and stopped on a layer of fine sand about 6 ft above the compact clay layer. Before deciding upon the stopping of wells at the higher elevation, the wells were to be tested to $1\frac{1}{2}$ times the design load. Plugging the wells and conducting full-scale tests meant expense, delay and damage to the wells when the plugging was removed, if the tests showed further sinking to be necessary. Therefore, plate bearing tests on the foundation soils have been conducted and the test results interpreted to the prototype. Also, tests of soils below the foundations for ascertaining the settlement were made. It was found that differential settlement of 4.4 in. might result. Therefore, to make good the difference in levels of the superstructure consequent on the differential settlement that might occur and to avoid damage to the bridge, provision has been made for lifting by jacks.

The Paper describes in detail the design of the mix, the procedure adopted for control of quality of the concrete for precasting work, the precasting of the beams, and the launching of the beams from the casting yard to their positions on the piers. The girders, each of which weighed 85 tons, were transported to position, using the girders already laid in position, by specially manufactured cars and employing a V-shaped launching girder 235 ft long. Salient features of the design of the superstructure, calculations of stresses and elongation in the cables, the precautions to be observed to reduce friction between the sheaths and the cables, and practical hints on safety measures to be observed during prestressing work, are given in detail.

1. INTRODUCTION

1.1. The Coleroon Bridge at Anaikaranchatram on Chidambaram-Sirkali Road connects Tanjore district, the granary of the south, with the neighbouring district of South Arcot. As early as the year 1853, an arched bridge of 42 spans of 50 ft each was built near Anaikaranchatram on this road, but it collapsed during the unprecedented floods of the year 1903. The collapse appears to have been caused by the blocking of one of the spans near the south bank by a huge *Arasu* tree uprooted and floated down by the floods.

2. THE SITE

The site of the old road bridge was the best available. It offered a square crossing across the river with high, well maintained and straight flood banks for a considerable distance on either side. There was no need for costly new approaches. But since the remains of the old bridge would form an obstruction during construction, a site 135 ft upstream of the old bridge and 1,115 ft upstream of the existing railway bridge was selected. The centre line of the proposed bridge was made parallel to that of the railway bridge, Plate I.

3. HYDRAULIC PARTICULARS

3.1. The river in its upper reaches has a number of reservoirs and anicuts and, therefore, is well controlled. The reservoir at Mettur provides for a maximum discharge of 3,00,000 cusecs. Deducting the normal draw-off of 30,000 cusecs for irrigation purposes in the lower reach, the net discharge comes to 2,70,000 cusecs.

3.2. After the construction of the Mettur Dam, the maximum observed discharge over the Lower Anicut, which is only 20 miles upstream of the bridge site, is 1,83,383 cusecs. Allowing an increase of $33\frac{1}{3}$ per cent, a maximum discharge that can be expected is 2,44,510 cusecs. Making a further addition of about 25,000 cusecs for the interim catchment and draw off, the net discharge is only 2,70,000 cusecs.

3.3. The discharge at site of the proposed bridge as calculated from Kutter's formula for the recorded maximum flood level is 2,30,500 cusecs. The railway bridge nearby has been designed for a maximum discharge of 2,70,000 cusecs.

3.4. Linear Waterway and Free Board

The linear waterway of the railway bridge is 2,100 ft consisting of 14 spans of 150 ft each. While the waterway is the same as that of the road bridge which had failed, the free board of the railway bridge is considerably more. So, a free board of 5 ft was provided in accordance with the recommendations of the Indian Roads Congress.

4. SELECTION OF THE SPAN

The approximate cost for 30 vents of 70 ft Tee-Beam bridge of reinforced concrete worked out to Rs 25.63 lakh and that for a Bow String Girder bridge with 14 spans of 150 ft each came to Rs 24.22 lakhs. Comparison of costs of structures with 50, 60, 100, 125, 175 and 200 feet bow string girders showed that the 100 ft span was the most economical. But the difference between the costs for spans of 100 and 150 ft was not appreciable. Provision of 150 ft spans would reduce the number of wells to be sunk and the period of construction, and also conform to the spans adopted in the railway bridge nearby. A 150-ft span with bow string girders was decided upon. To prevent formation of eddies and consequent scour, the centre lines of the piers of the new bridge were kept in line with the corresponding ones of the railway bridge.

5. SOIL PROFILE AT THE SITE

The soil profile at the site of the bridge, Plate II, shows successively layers of loose sand, sandy loam, clayey loam then a thin layer of medium to fine sand from about 50 to 60 ft below bed level, followed by compact clay and hard compact clay. The soil layers up to the compact clay display a somewhat erratic pattern with a number of pockets of different soils in the different strata.

6. FOUNDATIONS

6.1. Depth of Foundations

For design purposes the depth of scour was calculated to be 50.11 ft below the maximum flood level. The foundations were to be taken to levels below elevation - 30.59 from consideration of scour. But hard compact clay was met with between elevations - 62.0 and - 85 and it was decided that the foundations should be taken 5 to 6 ft into hard compact clay. The actual depth at which the foundations might be stopped was to be altered, if necessary, during execution of the work after conducting suitable load and other tests on the proposed foundation soils.

6.2. Nature of Foundations

In order to reduce the pressure on the foundation soil to a minimum, the choice was restricted to rectangular and twin octo-

gonal type of wells and to facilitate sinking for the large depth rectangular wells were preferred. Three rectangular wells of size 11 ft by 12 ft 6 in. were provided for each pier with clear spaces of 2 ft 6 in. in between. For the abutment, three wells of size 13 ft 6 in. by 15 ft were provided. The wells for the wings were of size 9 ft by 13 ft with a spacing of 2 ft 6 in. as for piers.

6.3. Well Kerbs

The kerbs were 2 ft 6 in. thick and 3 ft deep and were of 1 : 2 : 4 concrete suitably reinforced. The cutting edge of the kerbs was of mild steel angles 6 in. by 6 in. by $\frac{1}{2}$ in. with $\frac{3}{4}$ in. diameter mild steel bond rods running right through the centre of the steining. Photo 1.

6.4. Steining

The steining was of 1 : 3 : 6 concrete with $1\frac{1}{2}$ in. size stone aggregates. The thickness of steining was 2 ft for the top 40 ft and 2 ft 6 in. for the remaining depth below. Photo 2.

6.5. Plugging of Wells

The bottom plugging of 1 : 2 : 4 concrete was 6 ft thick laid in two layers of $3\frac{1}{2}$ and $2\frac{1}{2}$ ft with an interval of three days between the laying of the two. The top plugging of 1 : 4 : 8 concrete was 2 ft thick with $1\frac{1}{2}$ -in. aggregates and was laid over sand filling suitably laid and rammed.

6.6. Capping Slab

The capping slab over the 3 wells of the pier was 42 ft 6 in. by 11 ft by 2 ft thick. It was designed to transmit the load directly to the steining neglecting the area covered by the concrete plugging in the shaft of the well. The capping slab over the wells of the abutments and wings was a continuous slab 2 ft thick suitably reinforced.

6.7. The Substructure

The piers and abutments were of brick masonry in cement mortar 1 : 4 plastered with cement mortar 1 : 4.

6.8. Sinking of Wells

The first 12 ft of sinking through the top sand stratum was done by divers without the use of helmets. Thereafter excava-

tion by manual methods was difficult and slow. Dredging with a grab (capacity 3 cu. ft.) operated by means of a 2-ton winch was successful in the remaining depth of the sand layer. But the grab was ineffective in layers of sandy and clayey loams, and recourse was taken to manual excavation under water with the aid of helmets and compressed air.

6.9. Sinking through the top sand layer was done without any external loading on the wells. Wells were loaded externally with kentledge of sand-cement blocks to overcome resistance to sinking in greater depths. Where external loading was of no avail, the water level inside the well was depressed by pumping and the artificial difference in the head of water loosened the grip of the soil owing to flow of water down the outside of the shaft and the wells sank.

6.10. When the bottom of the wells reached an elevation of -50 to -55 ft, a sand layer of varying thickness was met with. Load tests were conducted on this soil layer at selected elevations under a number of piers and these results were supplemented with laboratory tests on the clay samples from deeper elevations. The wells were then rested on the sand layer at suitable elevations and were plugged. The details of the nature of the load and soil tests conducted and the interpretation of the results are given in sections 9 and 10.

6.11. Soil Blows During Sinking

During the sinking operations soil blows occurred, when a thin soil layer, relatively less permeable than the stratum below, was met with and or when pockets of loose soil materials were present in the vicinity of the bottom of the well. Photo 3 shows the subsidence of the soil at top when a blow occurred in the well. As a principle, the sinkers would excavate the soft spots in the bottom of the wells towards the close of each stage in the dredging operation to reduce the possibility of a premature blow. From experience the sinkers could detect the signs of an impending blow and care was taken to prevent a tilt due to the blow. Pumping was well controlled or stopped if necessary.

6.12. When one of the wells of pier No. 9 reached elevation -52.00 in clayey loam and dredging at the bottom was completed, it was too late in the day to proceed with the work. The next morning it was noticed that a heavy blow had occurred filling the well with soil up to elevation -25.00 feet. The well was cleared of the blown up soil up to the top of the kerb level. It was

reported by the divers that water was bubbling through, as in a spring, from the side where the blow had occurred. Slight excavation of the soil on the side where the blow had occurred showed continuous filling in of soil from below and the sides. The load at the top of the well was then increased from 30 tons to 60 tons and sand bags were laid over the soft spot at the bottom of the well. Excavation was continued in the other portions of the well, taking care not to disturb the soil below the sand bags. The sand bags were found to be sinking down slowly but no blow occurred. The sand bags were then pushed well into the bottom and sides against the kerbs, and the rest of the bottom of the well was cleaned. The signs of a blow were now completely absent. The water level in the well was then depressed and the well sank. The soil at the bottom of the well was examined by the sinkers and it was reported that the loose soil pocket on the side of the well where the blow had occurred had passed. The sand bags were removed and further sinking was continued. The soil met with was the clayey loam occurring at elevation -52.00 . The sand stratum was touched only at elevation -54.30 . Plate II shows the presence of sand at elevation -51.00 , -54.30 and -57.8 under piers 8, 9 and 10 respectively, i. e. the sand layer slopes downwards from the eighth pier to the tenth, leaving a thinner layer of clayey loam on the left side of pier No. 9 as shown in Fig. 1. Evidently this variation in the thickness of the clayey loam under the well had caused the blow.

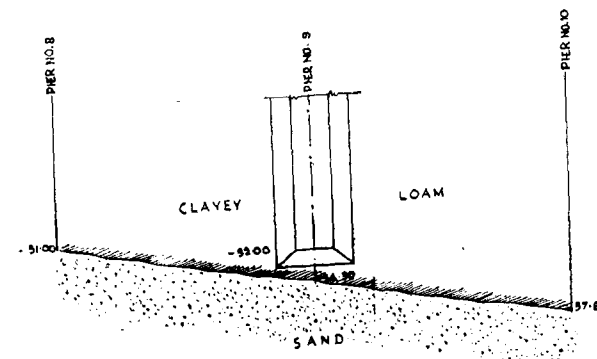


Fig. 1. Soil Profile below Pier.

6.13. Tilts

The method adopted to remove tilts consisted of driving casuarina piles under the cutting edge of the lower side, excava-

ting the soil on the higher side, and sinking by loading the well suitably. Sometimes the water level in the well was depressed.

6. 14. Difficulties due to logs of wood, skeletons of animals, etc., getting under the kerb of the wells were experienced during sinking. The obstructions were removed by divers.

6.15. During the sinking of one of the wells for pier No. 11, one fatal accident occurred, due to the presence of organic poisonous gas released during excavation. To avoid recurrence, fresh water was pumped into the well, the water level was raised, and the poisonous gas eliminated.

7. INSTALLATION OF SCOUR METERS

At present, calculation of depth of scour is generally based on empirical formulae. So, for collecting data under field conditions, a scour meter was installed in pier No. 12. The scour meter consists of four mild steel tubes, 4 in. in diameter, with their lower ends set at different depths. The pipes are attached to the well steining at appropriate levels and sunk with the wells. Inside each tube is a $1\frac{1}{4}$ in. diameter steel rod connected with a light flexible wire to a drum. Any scour below the bottom of the tube will be revealed and recorded by a downward movement of the rod and wire which will sink with bed as it is scoured during a flood. Details of the cantilever platform for installing the scour meter are shown in Plate III.

8. RIVER TRAINING

The south side of the Coleroon river bed had been heavily scoured up to an elevation of 17.00 and the deep running course of the river was on this side. Temporary groynes at an inclination of 35 degrees to the direction of flow were constructed, during the period July—September 1949, commencing from a distance of 1,700 ft upstream of the bridge site at intervals of 200 to 300 ft. In a period of three months after the completion of these groynes, heavy silting in the scoured area was observed and then **Korukkai** and **Nanal** grasses were planted to prevent bank erosion. The construction of the groynes resulted in the raising of the bed level in the vicinity of the south approach to about elevation 0.00. But the main channel of flow continued to hug the south bank. During the heavy floods of August 1954, by which time the south approach with its protective revetment had been raised to about elevation 8.00, observations of stream flow emphasised the extremely exposed nature of the south approach and the need for further protective measures.

9. TEST ON FOUNDATION SOILS

9.1 Considerations for Fixing Foundation Level

The level at which a bridge should be founded has to be fixed taking into account the following three essential requirements:

- (a) The foundations are below the maximum scour depth.
- (b) The foundation soil and the layers of soil below it have adequate bearing capacity and are safe against shear failure.
- (c) The deformation of the base of the structure due to unequal settlements is not large to damage the structure.

The foundations of the bridge have been taken below the maximum scour level, namely,—30 ft to about elevation—50 ft and so condition (a) mentioned above is satisfied. The requirements (b) and (c) mentioned above were to be determined on the results of suitable load tests in the field.

9.2. Limitations of Test Loading Wells

The original proposal was to test load some selected wells to $1\frac{1}{2}$ times their full design load, viz., 540 plus 270 or 810 tons. In order to conduct these full scale tests, the wells would have to be plugged. If the foundation soils stood the tests successfully further sinking was not necessary and work on the substructure could be proceeded with. On the other hand, if the tests showed poor results and if the wells were to be taken to deeper levels, the plugging used for the load test would have to be blasted or broken open and all the plugged materials removed. This might damage the steining. An alternative was to have the concrete plugging of a lean mix purely for test purposes and remove it by a suitable method after the completion of the test irrespective of the nature of test results.

9.3. Application of Kentledge Meant Delay and Expense

A kentledge of 810 tons was necessary to conduct the full scale load tests. No facilities were readily available at the bridge site to build up such a kentledge. Transport of scrap iron rails, 24 ft to 36 ft in length, from Golden Rock near Tiruchirappalli and back besides involving difficulties due to wagon shortage and delay, would cost Rs 24,000. In addition,

the cost for conducting each load test was roughly about Rs 6,000. So, the alternative method of conducting the tests on the soil using a bearing plate of a convenient size and correlating the results to the probable behaviour of the prototype was adopted.

9.4. Load Tests and Their Significance

Describing the limitations³ of load tests on bearing plates, D. W. Taylor says :

"Load tests have often the appearance and reputation of being a scientific approach and correctly so. The use of a scientific approach is one which carries much weight with many non-scientific men and there lies some danger. The proper interpretation of loading tests is far from a simple matter. Loading tests which are correctly interpreted offer a truly scientific attack to the problem but unless the scientific aspect extends to the interpretation, the use of the tests may be more harmful. Thus, the inherent limitations of such a process must be adequately realised in taking up the method."

The size and shape of the bearing plate, the encasement or boxing of the loading structure, provision of surcharge, rate of loading, the water level during the test, etc., affect the results significantly. But whatever be the method adopted, the results reflect the character of the soil within a depth approximately equal to twice the width of bearing plate only. Consequently, if changes occur in the type of soil in lower layers, the results may be misleading. (Fig. 2). If the relative density of sand or the

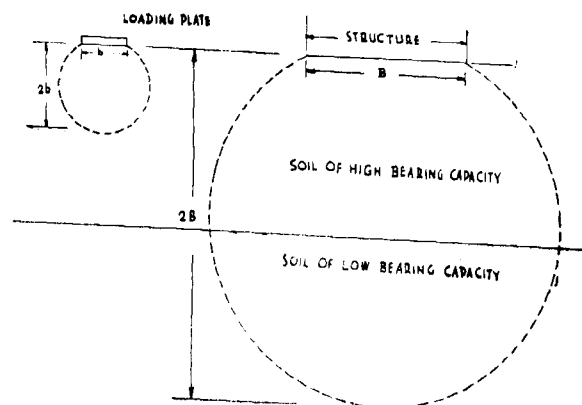


Fig. 2. The Case in which Surface Loading Test is likely to be misleading.

consistency of clay varies between the load test level and a significant depth below i. e. about twice the width of the proto-

type, additional load tests must be conducted at one or two intervals within this depth and the allowable load should always be selected on the basis of the unfavourable test results⁴. On the basis of a number of small scale load tests and settlement observations on buildings, Terzaghi gives certain empirical relationships between the settlements of the bearing plates in what he terms as 'standard' load test and the probable settlement of the prototype⁴.

9.5. Standard Load Tests

In the standard load tests (for sands) a bearing plate one foot square is located at the bottom of a test pit at least 5 ft square. The bearing surface is at the level of the base of the footing. The load is applied in increments of 200 pounds and increased to at least 1.5 times the estimated allowable soil pressure. The settlement measuring device employed permits readings to an accuracy of 0.01 in. ⁴.

The load-settlement curves are then plotted and the results analysed for the bearing capacity and the permissible differential settlement, taking into consideration the nature of soil profile, soil properties, type of superstructure, etc.

9.6. Load Tests at the Coleroon Bridge

The load test conducted at the Coleroon bridge differed considerably from the standard test described above and embodied some of the recommendations made in the Boston Building Code¹. Laboratory tests on soil samples obtained from selected locations were conducted for supplementing the field load tests.

9.7. Arrangement for the Test.

The loading device consisted mainly of a bearing plate assembly attachable to the bottom of a stanchion which was connected at its top to a loading platform and a measuring device to record settlements accurately.

9.7.1. The bearing plate assembly consisted of a mild steel plate, 24 in. in diameter and 1 in. thick, rigidly welded into a box unit with a mild steel plate, 18 in. by 18 in. by $\frac{1}{2}$ in., by means of plate ribs, 6 in. high. The 18 in. square plate was welded to a spacer unit and a threaded rod to facilitate connection of the bearing plate assembly with the base of the stanchion, A, B and C, Plate IV.

9.7.2. The stanchion consisted of 4 equal angles, 6 in. by 6 in. by $\frac{1}{2}$ in., located at the four corners of a square of 2 ft

sides. The sides were laced with mild steel flats, 2 in. by $\frac{1}{2}$ in. For convenience in transporting and handling, the stanchion was made up of five units varying from 17 ft to 6 ft in length. There were three numbers of length 17 ft (No. 1, 2 and 3), one of 10 ft (No. 4) and one of 6 ft (No. 5). All, except the base stanchion (No. 1), were welded with mild steel angles, 6 in. by 6 in. by $\frac{1}{2}$ in., at the top and bottom for providing a suitable bearing surface for transmitting the loads when inter-connected. The base stanchion was welded at the top with mild steel angles, 6 in. by 6 in. by $\frac{1}{2}$ in., and at the bottom to a box unit with mild steel angles and plate ribs with a square opening at the centre, D in Plate IV, to take the spacer unit of the base assembly. In the test position the bottom plate of this box unit, D, rested on another one of its size attached to the bearing plate assembly, B. Each stanchion unit could be rigidly connected to the next one by means of 4 bolts near the four corners of the surfaces of contact and 4 mild steel angle splices, 6 in. by 6 in. by $\frac{1}{2}$ in. and 4 ft in length. The bottom face of each of the top stanchions was provided with two studs on opposite sides and the corresponding sides of the top face of the stanchions had two holes. This facilitated easy alignment during erection, Plate IV.

9.7.3. The loading platform consisted of 4 mild steel angles of 6 in. by 6 in. by $\frac{1}{2}$ in. connected together rigidly such that the centre portion could be conveniently seated on the top face of the stanchion. The device for measuring settlement consisted of a knife-edge rigidly attached to the sides of the topmost stanchion, a lever arrangement, a knife-edge rest and a pointer with a scale attached to an independent support. The knife-edge was attached rigidly to the frame of the stanchion by means of two sets of galvanized iron pipes and a connecting device, Plate IV. The movement of the knife-edge rest was magnified by the lever 10 times to read to an accuracy of 0.005 inch. The attachment holding the fulcrum was capable of side movement which facilitated lateral adjustment for seating etc.

9.8. Erection of the Loading Device

9.8.1. The surface of the foundation soil in the well was cleaned and made level. A four-legged derrick of casurina posts of 9 in. girth was erected over the well. Depending on the elevation of the soil to be test loaded, combinations of the stanchion units were selected. The bearing plate assembly was attached to the bottom of the base stanchion by means of the 4 bolts and the splices at the four corners. The top of the stanchion was attached to the hook of a movable pulley-block

arrangement. By operating a 2-ton winch, the stanchion was lifted, suitably moved to position and lowered into the well. The heavy weight of the stanchion had a considerable dragging and overturning effect on the derrick. The derrick was strengthened with wooden lacing. To control the drag and swing of the stanchion a second winch was used to release the stanchion slowly as it was being lifted off the ground with the first winch. This arrangement gave satisfactory results.

9.8.2. When the top face of the stanchion was about 2 ft above the steining of the well, an R. S. joist 6 in. by 10 in. and 13 ft long was introduced into the stanchion horizontally and the two were lowered down so that the joist rested on the well steining and supported the stanchion. Finer adjustments were then made to bring the stanchion at the dead centre of the well and the hook attached to the top of the stanchion was released.

9.8.3. The second stanchion was lifted and lowered gently till it touched and rested square on the top face of the base stanchion. The studs at the bottom surface of the second stanchion facilitated sliding and vertical alignment. The side splicings were then fitted and tightened followed by the tightening up of the bolts near the four corners. In the beginning, the side splicing connections were not quite flush and rigid. This was set right by filing the splicings in the appropriate places. The assembly of the two stanchions was lifted up gently and the joist was removed from the base stanchion. The stanchion assembly was then lowered into the well and just before the top of the second stanchion reached the top surface of the well steining, the joist was again introduced into the stanchion so that, on lowering, the stanchion was supported by the joist. The required number of stanchions was thus added to make up the depth of the well. The joist was itself supported on two 10-ton hydraulic jacks which, in turn, were seated firmly on two brick pillars with a mild steel plate $\frac{1}{4}$ in. thick on top. By operating the jacks, the load transmitted by the stanchion to the soil below was controlled as described later.

9.8.4. In the initial position the jacks were lowered and the bearing plate was allowed to rest on the foundation soil. Side struts of casurina stems were fixed and ropes were tied to the stanchion at top to keep it vertical at the centre of the well.

9.9. Surcharge

The surcharge provided around the bearing plate was based on the recommendations of the Boston Building Code and

determined from the relationship, Fig. 3.

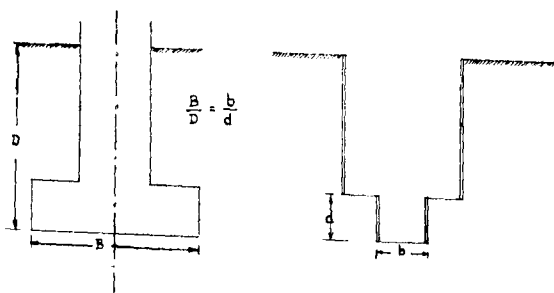


Fig. 3. Height of Surcharge for Plate Bearing Test.

$$\frac{B}{D} = \frac{b}{d}$$

where B = breadth of the well

D = depth of the well

b = breadth of the bearing plate

d = depth of surcharge for the load test.

Taking an average depth of $D = 55$ ft and width $B = 11$ ft,

$$\frac{B}{D} = \frac{11}{55} = 0.2$$

Therefore the surcharge for a 2 ft diameter plate

$$\frac{2}{0.2} = 10 \text{ ft}$$

On an average, the surcharge of 10 ft of sand was laid for the full area of the well and no boxing or encasement of the stanchion was done. All the tests were conducted below the water level.

9.10. Method of Loading and Observations

9.10.1. An ideal arrangement for controlling transmission of load to the bearing plate would be to erect a suitably loaded platform above the centre of the well and jack the stanchion against this platform at the desired rate. The load transmitted could be read with a pressure gauge attached to the jack. But jacks and pressure gauges of the required capacities were not readily available. So, two jacks were used. The procedure adopted for loading is described below.

9.10.2. The pointer in the deflectometer was set to zero and settlement due to the weight of the stanchion and platform was taken as zero. On completion of the tests a suitable correction for this error was made in the plotting of the load-settlement curves. The platform and the stanchion were suitably supported by means of the two hydraulic jacks so that the pointer continued to read zero. In other words, further settlement due to the initial load or self weight of the stanchion and platform, 1.14 ton per sq. ft., was prevented. A load increment was effected symmetrically on the platform and the load was raised to 2 tons per sq. ft. on an area of diameter 2 feet. The reading on the deflectometer was still at zero and the load increment was being taken up by the jacks and not by the stanchion and bearing plate assembly. The jacks were gradually and simultaneously released so that the load was transmitted to the soil without undue impact. This method ensured uniform load transmission to the bearing plate. Possible errors due to changes in the rate of load increment on the platform were avoided. Immediately after releasing the jacks, settlement observations were made at 30 seconds intervals for the first 5 minutes and 2 minutes intervals until there was no perceptible settlement for the 2 minutes interval. When settlement observations came to a close for this load, the jacks were operated suitably as before and further settlement was arrested. The platform was ready for the next load increment. The tests were conducted with load intensities of 2, 3, 4, 5 and 6 ton per sq. ft. on the bearing plate. The next load was 7.25 ton per sq. ft. which was the design load without allowance for buoyancy and skin friction. With this load, the settlement observations were continued until no perceptible settlement occurred for a period of 24 hours. The loads were then increased in four stages from 7.25 to about 14.5 tons per sq. ft. The readings for the three intermediate loads were recorded for a period of not less than 4 hours and for the final load until no perceptible settlement occurred for a period of 24 hours. The load tests were conducted in the wells of piers 5, 6, 7, 8 and 9, Photos 4, 5 and 6.

9.10.3. Slight variations were effected in a few cases in the time intervals for settlement observations and in the amounts of load increments. These changes would not appreciably affect the net results in the load-settlement curves. No tests were conducted at different elevations.

9.11. Results of Load Tests

The results of the load tests are plotted and the load settlement curves are given in Fig. 4 and 5.

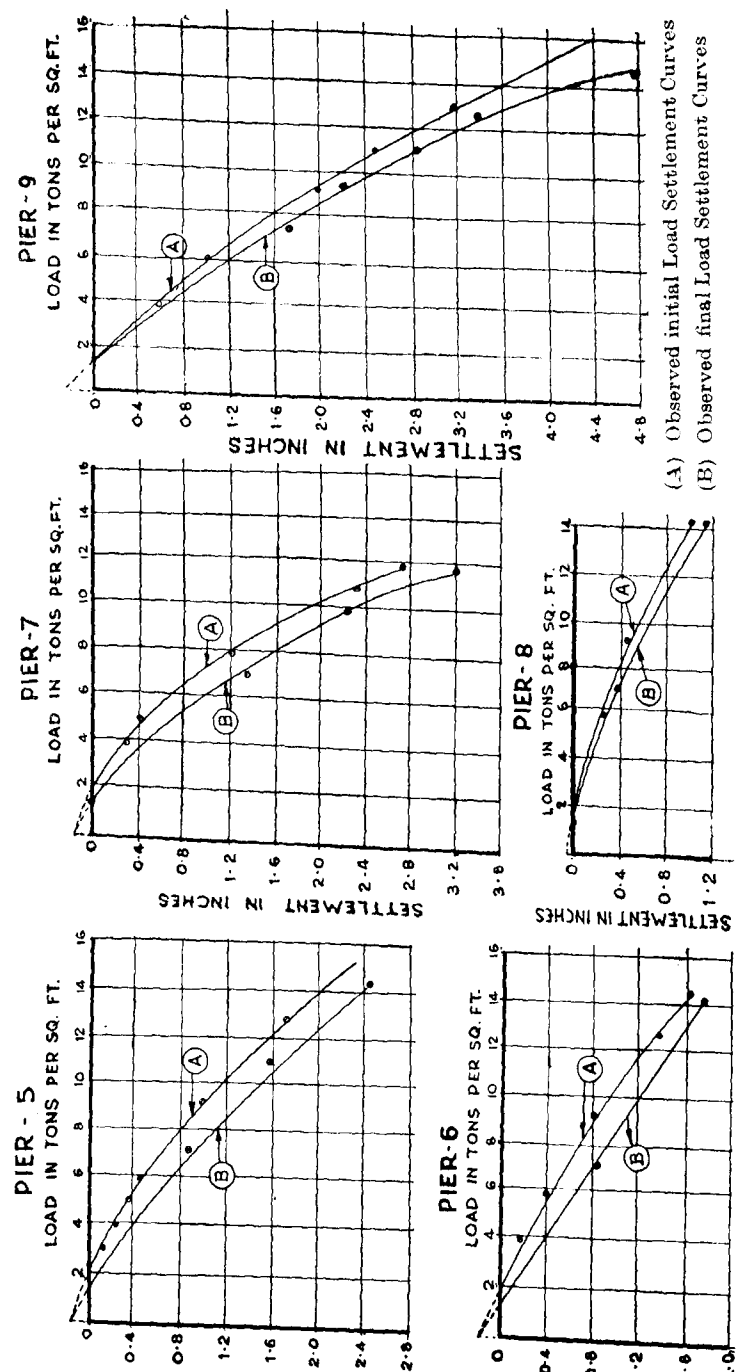


Fig. 4. Load Settlement Curves for Piers 5, 6, 7, 8 and 9.

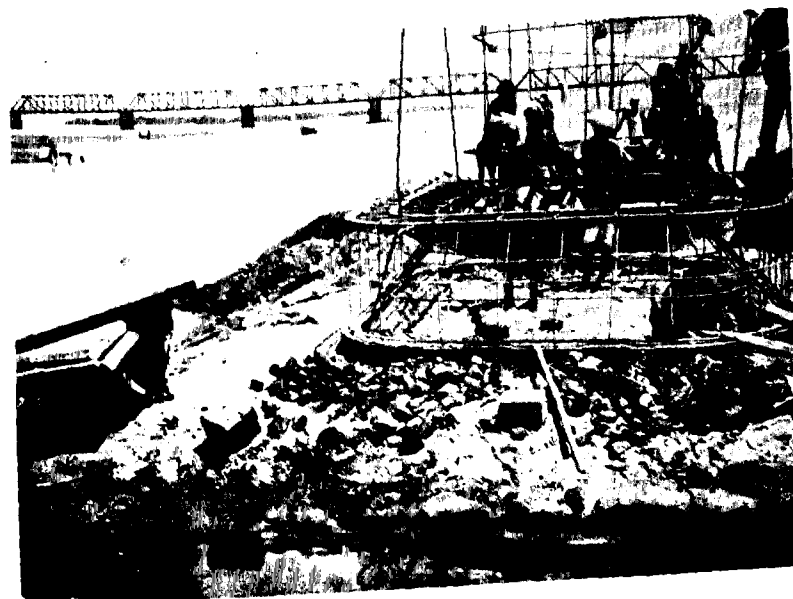


Photo 1. Reinforcement of Well Kerb.

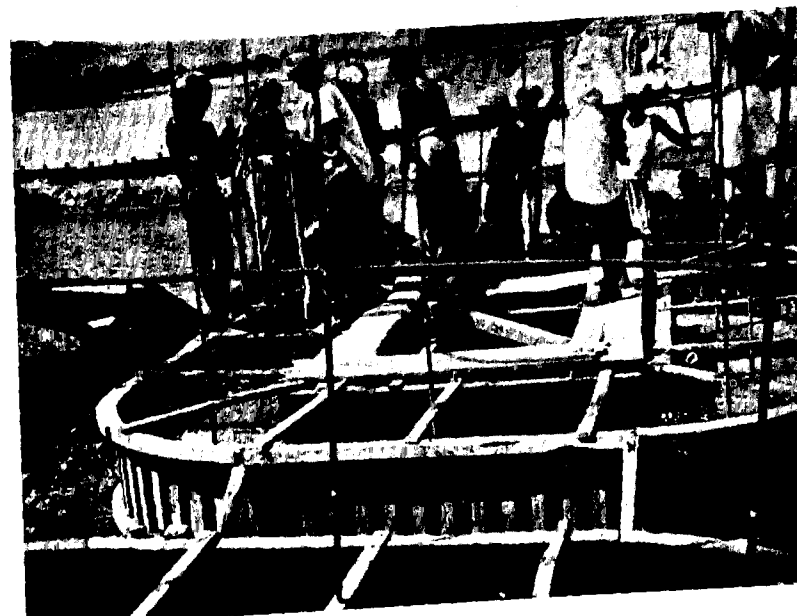


Photo 2. Formwork for Steining.



Photo 3.
Subsidence
outside the well
when a sand
blow occurred.

Photo 4.
Stanchion is
ready for
loading.

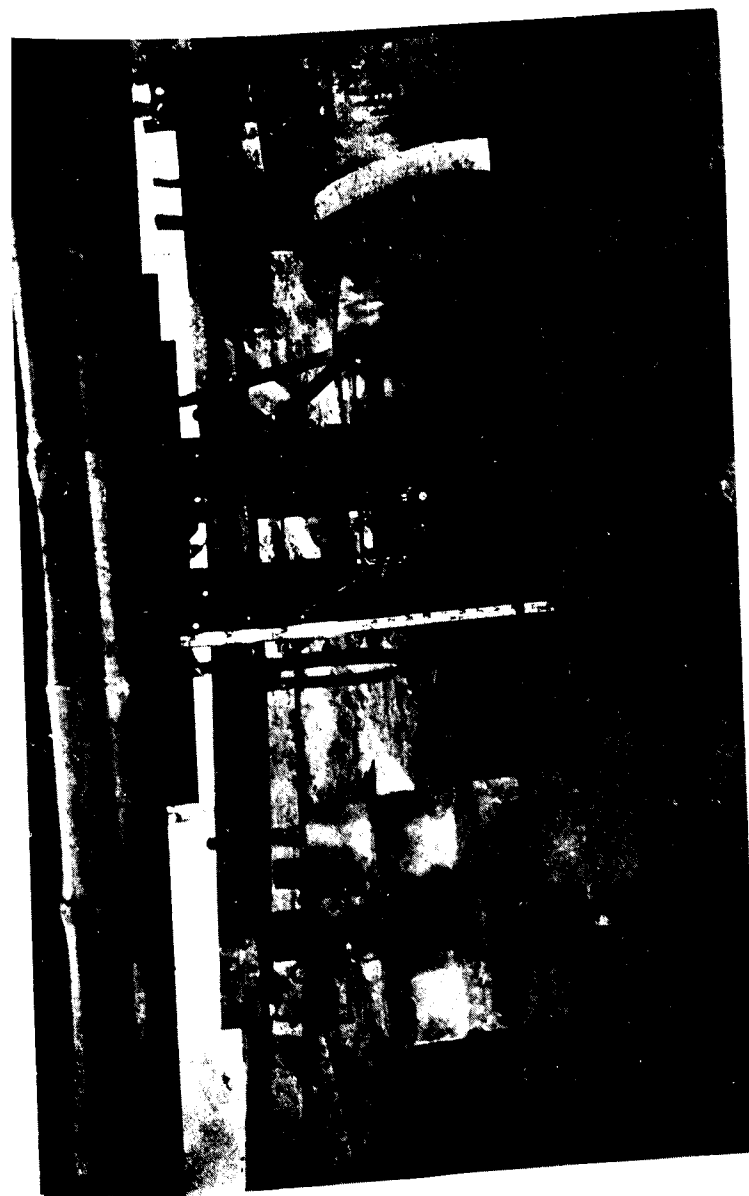
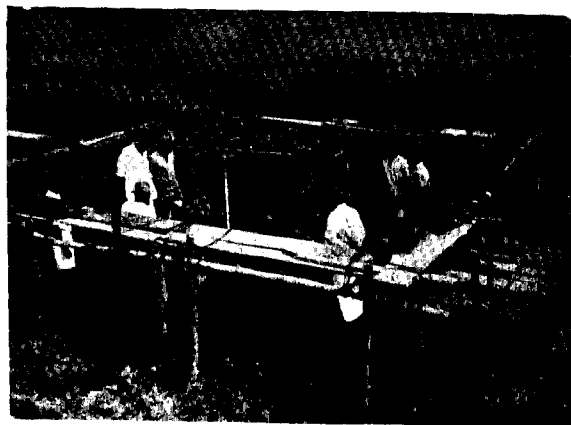
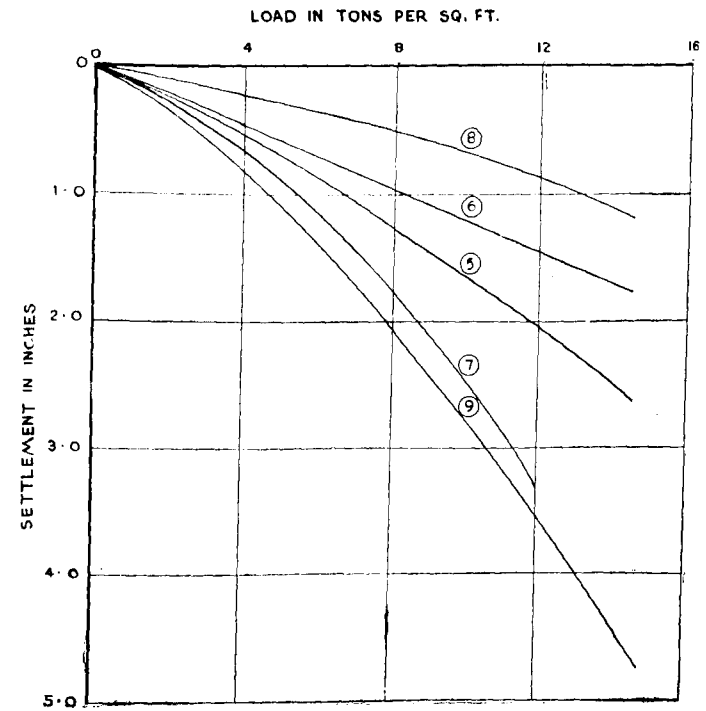


Photo 5. Load applied over Stanchion and settlement under observation.



Photo 6. Removal of Stanchion after Test.



Ringed numbers refer to Piers for which tests were conducted.

Fig. 5. Relation between Load and Settlement of 24 in. diameter Bearing Plate Resting on Sand at the Foundation Level with a Surcharge of 10 ft of Sand.

9.12. Soil Tests

Tests were conducted in the laboratory on a limited number of disturbed and undisturbed soil samples obtained from foundations of piers 5 and 9. The samples were tested for mechanical analysis consistency limits, natural moisture content, natural density, specific gravity, and direct shear.

10. INTERPRETATION OF LOAD AND SOIL TESTS

10.1. Design Load Considerations

10.1.1. The original design was made without making any allowance for buoyancy and skin friction. During the sinking of wells it was observed that considerable skin friction developed

and special effort was necessary to sink the wells. Allowing for a scour depth of 30 ft, the soil below the scour level was counted for a fraction of the skin friction actually observed during sinking ⁴.

10.1.2. Being subject to tidal action the water level at the bridge site is never below elevation 0.00. In accordance with Terzaghi's experiments and findings on allowable hydrostatic uplifts for structures below water table ¹, full hydrostatic uplift was allowed for the submerged portions of the foundations. Making these allowances, the load due to prestressed superstructure and foundations on the soil worked out to about 4.30 ton per sq. ft. when the foundation was carried to an elevation of about 55 feet.

10.2. Factor of Safety for Bearing Capacity

For the safe allowable pressure on the foundation soil a factor of safety of 3 is normally recommended and for special conditions, an absolute minimum of 2 may be permissible ⁴. Foundations on sand, and especially deep foundations, rarely fail by shear. A minimum factor of safety of 3 is commonly obtained in sand foundations and the allowable soil pressure on sand is exclusively determined by settlement considerations. On the other hand, in clay soils, due consideration is to be given for bearing capacity and settlement. The principles applicable for sands are applied for cohesionless coarser silts and those mentioned for clays are applied for silts showing plasticity.

10.3. Bearing Capacity of the Foundation Soils of the Bridge

10.3.1 **Load Test.** The load-settlement curves for the load tests were plotted under three groups, viz.,

- (i) Initial load-settlement curve with values corresponding to a duration of time when no perceptible settlement occurred for a continuous period of 2 minutes, Curve 'A', Fig. 4.
- (ii) Final observed load-settlement curve with values corresponding to a duration of time when no perceptible settlement occurred for a continuous period of 24 hours, Curve 'B', Fig. 4. The readings at the commencement of the load test and at loads of 7.25 and 14.5 tons per sq. ft. are the actual observed points during the test and the curve was interpolated suitably taking curve 'A' as a guide.
- (iii) Final corrected load-settlement curve wherein allowance is made for the probable initial settle-

ment due to the self weight of the loading device, Fig. 5.

10.3.2. No allowance was made in these curves for the loosening or heaving up of the foundation soil due to excavation of the overburden at top, nor was the effect of not encasing the stanchion at the bottom taken into account. The discussions in the succeeding paragraphs are based on values obtained from the curves given in Fig. 5.

10.3.3 None of the load-settlement curves shows a sharp yield point in the load test. The curves for piers 7 and 9 show a definite tendency for a local shear failure ³ indicating the presence of a relatively soft or loose type of a soil, while those for piers 5, 6 and 8 show the presence of a medium to dense type of sand. The failure in the latter case would have been by shear if the tests had been continued further. Allowing a factor of safety of 3 in all cases, except for pier 7, the safe allowable pressure against shear failure is easily more than 5.00 tons per sq. ft. In pier 7, while the total test load was not increased to a value more than 12 tons per sq. ft., the trend of the curve shows that the ultimate bearing capacity may be about 12 tons per sq. ft. Allowing a factor of safety of 3, the safe allowable pressure is 4 tons per sq. ft.; with a factor of safety of 2.5, the allowable pressure is 4.8 tons per sq. ft.

10.3.4 These safe allowable pressures hold good for the bearing plate 2 ft in diameter when the layer of the foundation soil is not less than 4 ft thick, i.e., twice the width of the plate. Its application for the prototype is valid only if the soil is homogeneous to a depth of about 2 times the width of foundation, viz., about 22 ft below the test load level. The soil profile shows variation in the soil types and for such cases it is necessary that additional load tests be conducted at suitable deeper elevations. This was not done in the field due to practical difficulties and other exigencies.

10.4. Laboratory Tests

10.4.1. The results of load tests show that the sand layer occurring under piers 5 to 9 possesses satisfactory bearing capacity and this may be taken to be true of the layer of the same type of sand under other piers and the abutments. The safe allowable pressure on the clay layer occurring below the sand layer was determined from the laboratory tests.

10.4.2. The soil at elevation—64.00 of pier 9 is a grey silty clay with a natural moisture content nearer to its liquid limit

than to its plastic limit and is to be taken when normally loaded, as susceptible to considerable settlement. The safe allowable pressure on the soil determined from Terzaghi's equation¹, with a factor of safety of 3, is 4.2 tons per sq. ft. while the actual pressure at that elevation as per Boussinesq's stress distribution is only 3.3 tons per sq. ft. This elevation is about 6 ft below the proposed foundation level of -57.63. The same silty clay extends to a depth greater than the significant depth below the load test elevation. Visual examination and check tests on additional soil samples show the presence of this clay under the abutments and piers. The hard compact clay underlying the compact clay is relatively stiffer and, therefore, possesses a higher bearing capacity.

From the foregoing discussion it is seen that:

- i. the sand layer on which the foundations were proposed to be laid possesses satisfactory bearing capacity, and
- ii. the load on the clay layer below the sand layer of 6 ft thickness is well within the safe allowable pressure.

10.5. Settlement Characteristics

10.5.1. Load Tests. There exists no simple or direct correlation between the settlement values obtained in the load tests and the probable settlements of the prototype. The available methods of analysis are based on empirical formulae or theoretical concepts based on a number of assumptions. The empirical formulae due to Terzaghi and Peck⁴ for sand and clay hold good for soils subjected to the standard load tests and take into account the variation in width between the bearing plate and the prototype but neglect the depth factor completely. A theoretical formula due to Taylor³ takes into account both the breadth and depth factors but the application of the formula is restricted to the initial straight line portion of the load-settlement curve obtained in the load test. In the present analysis the concept due to Taylor is adopted since the actual load test conditions compare fairly closely to the assumption made by him.

10.5.2. The total settlement under the prototype consists of the settlement due to the different layers of soils occurring from the foundation level to a depth equal to the significant depth, which is assumed as twice the width of the foundation, viz., 22 feet. The settlements due to the cohesionless top layer are obtained from the load tests and those due to the bottom cohesive

layer are roughly determined from available laboratory test results on the soil samples.

10.5.3. The relationship between the settlement values obtained from the load tests and the probable settlement under the prototype presupposes the existence of a homogeneous soil layer up to a significant depth for the given base width in both cases. For purposes of analysis, such an assumption is, therefore, made in the determination of the settlement values for the sand layer and to these values are added the settlements due to the bottom clay layer of a thickness equal to the significant depth minus the actual thickness of a sand layer below the load test level. The error is on the safe side.

10.5.4. Taking the equation given by Taylor for cohesionless soils³,

$$\frac{q}{c} = C \left(1 + \frac{2d}{b} \right) \quad \begin{array}{l} \text{where } q = \text{applied pressure} \\ c = \text{settlement} \\ C = \text{coefficient} \end{array}$$

the value of $\frac{d}{b}$ for the load tests is the same as that for the prototype and assuming that the final load-settlement curve is a straight line for loads from 0 to 4.35 ton per sq. ft., the settlement values for the prototype are directly read from the graph in Fig. 5.

10.6. Settlement

10.6.1. The three considerations in a settlement analysis are the total settlement, ultimate differential settlement, and rate of settlement. Of these, the rate of settlement is not a matter of primary concern since damage will eventually occur if the differential settlements are greater than those the structure can stand².

10.6.2. Settlement in sand soils is mostly complete during the period of construction, while in clay soils settlement takes place over a considerable time. No consolidation tests were performed on the clay samples underlying the sand layer and an approximate settlement prediction is made on the basis of Skempton's statistical analysis of the relationship between the values of liquid limit and compression index of the clay soils.⁴

10.6.3. If the wells were rested at such elevations that there was a layer of sand of uniform characteristics and thickness, the settlement would be only due to a less intensity of load coming on the underlying layers and if the ultimate and differential settlements are not high, the foundations could be rested at these elevations. The two soil layers below the sand layer have

been classified as 'compact' and 'hard compact clay' on the basis of visual identification and observations made during the sampling operations in the field. Laboratory tests were conducted on undisturbed samples of the top layer of compact clay, while in the case of the bottom layer these were done only on dried samples. The tests showed values of liquid and plastic limits quite close for both the soils indicating that the two soils might have similar settlement characteristics, provided the natural moisture content was the same in both. On the basis of the field observations, however, it has to be taken that the hard compact clay probably had a less moisture content. For purposes of settlement calculations, therefore, two extreme cases arise, viz.,

- (i) the bottom hard compact clay may be assumed to possess settlement characteristics very similar to those of the overlying compact clay layer, and
- (ii) the hard compact layer may be assumed to undergo little or no settlement.

10.6.4. In Plate II, the elevations at which the foundations of the piers and abutments have been actually laid, are shown underlined. The approximate settlements for the sand layer were obtained from the load test results and are given in Table I.

TABLE I

Approximate Settlement Values for Sand Layer under Piers 5, 6, 7, 8 and 9 as Determined from Load Tests

Under Pier No.	Settlement of fine to medium sand		
	Elevation (-ve)		Inches
	From	To	
5	52.30	69.80	0.60
6	53.30	65.00	0.50
7	54.20	59.80	0.80
8	53.90	60.70	0.25
9	57.90	63.60	0.95

TABLE 2
Approximate Settlement of Clay Layers Under the Piers and Abutments Determined from Laboratory Tests

Foundation Unit	Assuming same settlement characteristics for compact and hard compact clay				Settlement due to compact clay only			
	Elevations (-ve) of clay layer considered		Settlement in inches	Differential settlement	Elevations of compact clay		Settlement in inches	Differential settlement
	From	To			From	To		
North Abutment Pier No. 1	58.5	75.0	8.9	0.2	58.5	62.5	3.1	1.7
	58.0	74.0	8.7	1.5	58.0	64.1	4.8	0.4
	58.0	75.0	10.2	1.5	58.0	65.8	5.2	0.4
	65.0	79.5	8.7	1.5	65.0	71.2	5.6	2.1
	60.8	76.8	10.2	4.4	60.8	69.8	7.7	1.9
	62.2	74.2	5.8	4.4	62.2	74.2	5.8	4.4
	59.0	78.0	10.2	0.0	59.0	75.0	10.2	0.0
	60.7	76.2	10.2	1.5	60.7	76.2	10.2	1.5
	59.8	75.9	8.7	1.5	59.8	75.9	8.7	1.5
	63.6	79.6	10.2	0.0	63.6	79.6	10.2	3.5
	66.5	81.5	10.2	0.0	66.5	75.5	6.7	2.7
	62.3	78.0	10.2	1.5	62.3	75.0	9.4	0.7
	63.6	79.0	8.7	0.0	63.6	78.0	8.7	0.7
South Abutment	66.0	82.0	8.7	3.0	66.0	78.5	8.0	2.6
	67.0	81.0	5.7		67.0	80.0	5.4	

For purposes of these approximate calculations, the following average conditions of loads have been taken :

- Foundation elevation
Relief due to excavation
Foundation load for piers
Foundation load for abutments
- 57.5 ft
1.286 ton per sq. ft.
4.22 ton per sq. ft.
3.25 ton per sq. ft.

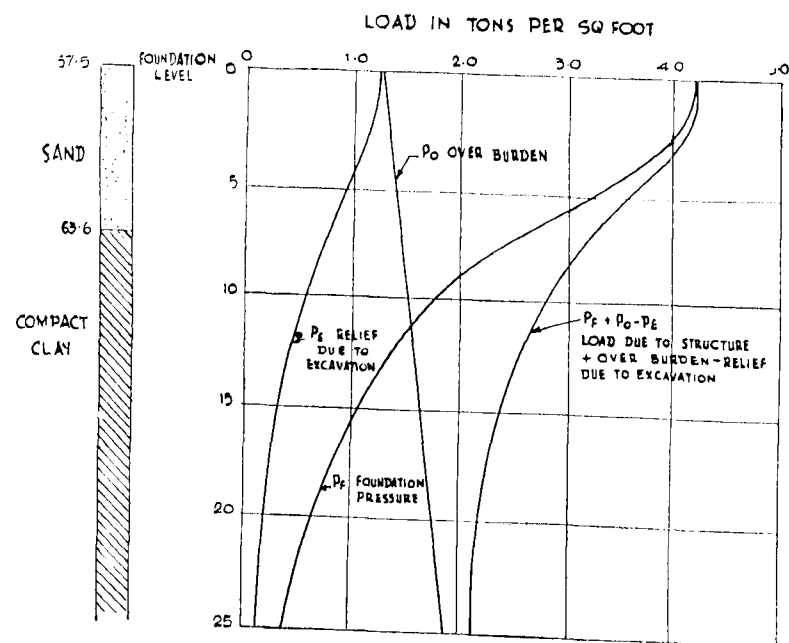


Fig. 6. Pressure Distribution below foundation of well for Pier No. 9.

PRESSURE DUE TO
STRUCTURE $q_1 = 4.22$

PRESSURE DUE TO
OVER BURDEN $q_2 = 1.266$

S. No.	DEPTH Z	LENGTH m/z	BREADTH mb/z	I_z	$q_1 \times I_z$ $\times 4$	$q_2 \times I_z$ $\times 4$	q_3 OVER BURDEN	NET LOAD
1	2	3.125	2.75	2.431	4.151	1.25	1.331	4.232
2	4	1.570	1.37	2.122	3.580	1.09	1.376	3.866
3	6	1.040	0.92	1.716	2.900	0.885	1.421	3.436
4	10	0.625	0.55	1.031	1.7400	0.534	1.511	2.717
5	15	0.415	0.365	0.597	1.010	0.308	1.624	2.321
6	20	0.312	0.28	0.362	0.61	0.187	1.736	2.159
7	25	0.25	0.22	0.238	0.385	0.123	1.849	2.110

SETTLEMENT CALCULATIONS

$$C_c = 0.009 (L.L. - 10)$$

$$= 0.009 (87 - 10) = 0.009 \times 77 = 0.693$$

$$S = H \frac{C_c}{1+e_0} \log_{10} \frac{p_0 + \Delta p}{p_0}$$

$$57.6 \text{ to } 63.6 \text{ SAND}$$

$$S_1 = 63.6 \text{ to } 67.6 = 4 \times \frac{0.693}{1+1.55} \log_{10} \frac{3.2}{1.48} = 0.364$$

$$S_2 = 67.6 \text{ to } 72.6 = 5 \times \frac{0.693}{1+1.95} \log_{10} \frac{2.62}{1.58} = 0.276$$

$$S_3 = 72.6 \text{ to } 77.6 = 5 \times \frac{0.693}{1+1.55} \log_{10} \frac{2.25}{1.7} = 0.165$$

$$S_4 = 77.6 \text{ to } 79.6 = 2 \times \frac{0.693}{1+1.55} \log_{10} \frac{2.15}{1.78} = 0.044$$

$$0.849 \text{ FT}$$

$$10.188 \text{ "}$$

H = Thickness of stratum; C_c = Compression index; C_o = Void ratio;
 p_o = Overburden pressure; Δp = Change in pressure.

Assuming that these settlements are complete during the period of construction itself, the differential settlements would be exclusively due to the settlements of the clay layers below. The probable settlement values due to the clay layers for the two extreme cases were determined assuming Boussinesq's pressure distribution and obtaining the value of compression index from its correlation to the liquid limit as given by Skempton^{4,5}. A typical pressure distribution and settlement calculation is shown in Fig. 6. Table 2 shows the approximate values of settlements due to the clay layers under the abutments and piers from the table. The maximum differential settlement is seen to be 4.4 inches. There is as yet no means of determining the permissible differential settlement for the foundations of a given structure; however, this depends upon a number of factors as the type of the structure, the nature of the construction material used and the time rate of construction and settlement². The superstructure in the present case is of prestressed concrete and is of an unbalanced cantilever type, capable of adjustment of the levels of the deck, if warranted. The required number of jacks has been purchased and suitable provision has been made in the superstructure to facilitate undertaking jacking operations

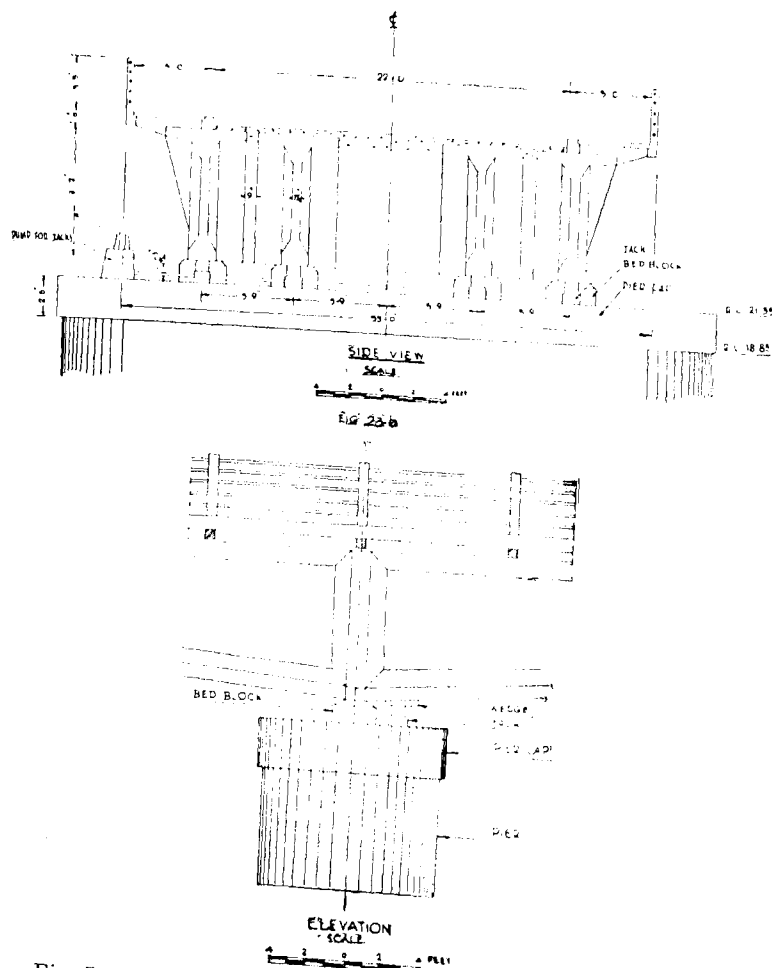


Fig. 7. Arrangement for Jacking if settlement occurred.

if need be. Fig. 7 illustrates the idea. A maximum differential settlement of about 4.4 in. was, therefore, not considered high⁶.

10.6.5. The load tests and the laboratory investigations showed that the soil layers below the load test levels possess adequate bearing capacity and are not susceptible to undue detrimental differential settlement. Advantage was thus taken of the presence of the sand layer near elevation -50 and the foundations for all piers and abutments were stopped at suitable elevations, leaving a sand layer of uniform thickness of about 6 ft under the wells. This has resulted in a saving of about Rs 1,00,000.

11. SOME FEATURES OF THE DESIGN

11.1. General

11.1.1. The tenders invited for the work in 1949 were found to be high. So, the construction of foundation and substructure was taken up departmentally through a piecework contractor. The foundations having been completed and the construction of the substructure well advanced by the middle of the year 1952, tenders were invited for the superstructure.

11.1.2. The departmental design for the bridge had provided R. C. Bowstring girders of 150 ft clear span since that was found to be the most economical at the time the project estimates were prepared. The lowest of the tenders received was for a prestressed concrete bridge and it was accepted.

Also, as against 1,487 tons of mild steel and 1,923 tons of cement required for the R. C. Bowstring Girder bridge originally proposed, only 152 tons of high tensile steel, 72 tons of mild steel and 1,560 tons of cement have been estimated for the prestressed design.

11.1.3. The contract amount for the superstructure, excluding the cost of mild steel, high tensile steel, cement and pipes for hand rails which are supplied by the department, is Rs 9,17,000. Including these materials, the cost of the superstructure works to Rs 13,18,000.

11.2. General Details of Design

11.2.1. The bridge is of the cantilever type, having a total length of 2,230 ft composed of 14 spans of 159 ft each between centres of piers, Fig. 8. The girders of the first span from the north bank rest on the north abutment and the first pier and cantilever

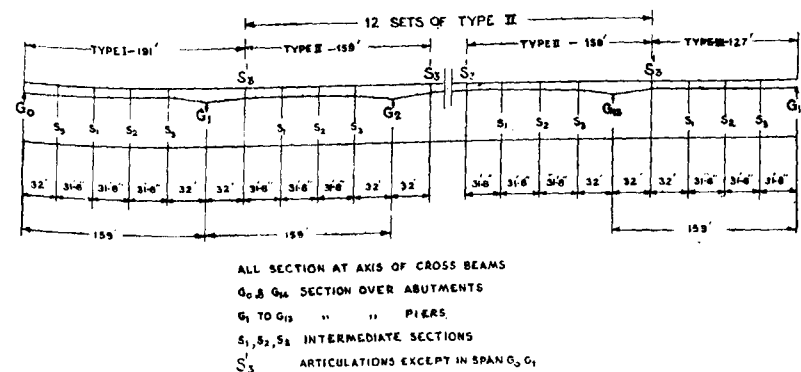


Fig. 8

into the second span to a length of 32 ft. The girders of the second span are supported on the ends of the cantilevers and on the second pier and cantilever into the third span for a length of 32 ft and so on until the 13th span. The girders of the 14th span are carried on the cantilevers from the 13th span and the south abutment. Thus, the beams can be divided into three main types, Fig. 8, as stated below:

Number of spans	Type	Simply supported length Ft	Cantilever length Ft	Total length Ft
1	I	159	32	191
12	II	127	32	159
1	III	127	Nil	127

11.2.3. The clear width of the bridge is 32 ft composed of a carriageway of 22 ft and two footpaths each of 5 ft wide. The wearing surface of the carriageway is of cement concrete 2-4-2 in. thick. The footpaths are of precast concrete slabs supported on a frame work of reinforced concrete which is cantilevered from the exterior longitudinal beams.

11.2.4. All the beams are precast on the shore, prestressed and launched by special launching devices. They are of T-section 6 ft 3 in. deep which is increased to 8 ft 2 in. at points of support over piers, *G* Fig. 8. Their flanges are 5 ft in width throughout. The thickness of the web of the beams is 6 in. except in the two bays on either side of *G* where it is increased to 7½ inches. In the upper end blocks (end of beam forming the upper part of hinge) and in the lower end blocks (tip of cantilever forming the lower part of hinge), the web thickness is 20 inches. The bottom flange is 20 in. throughout. Details are shown in Fig. 9.

11.2.5. The shape and dimensions of the beams have been so manipulated as to lend themselves to unit construction and the avoidance of a multiplicity of forms. Thus beams of types II and III may be considered as portions of type I. The 191-ft length of a type I beam is divided by diaphragms (cross-beam teeth) into 6 bays, two of 32 ft, three of 31 ft 8 in., and one of 32 ft, successively commencing from the lower end block. When the last bay of 32 ft is cut off, a type II beam having a total length of 159 ft is obtained. When the two bays on either side of *G* are cut off, the result is a beam of type III having a length of 127 ft. So one single form to suit type I beams with some special pieces for the end blocks can be used for the other types too.

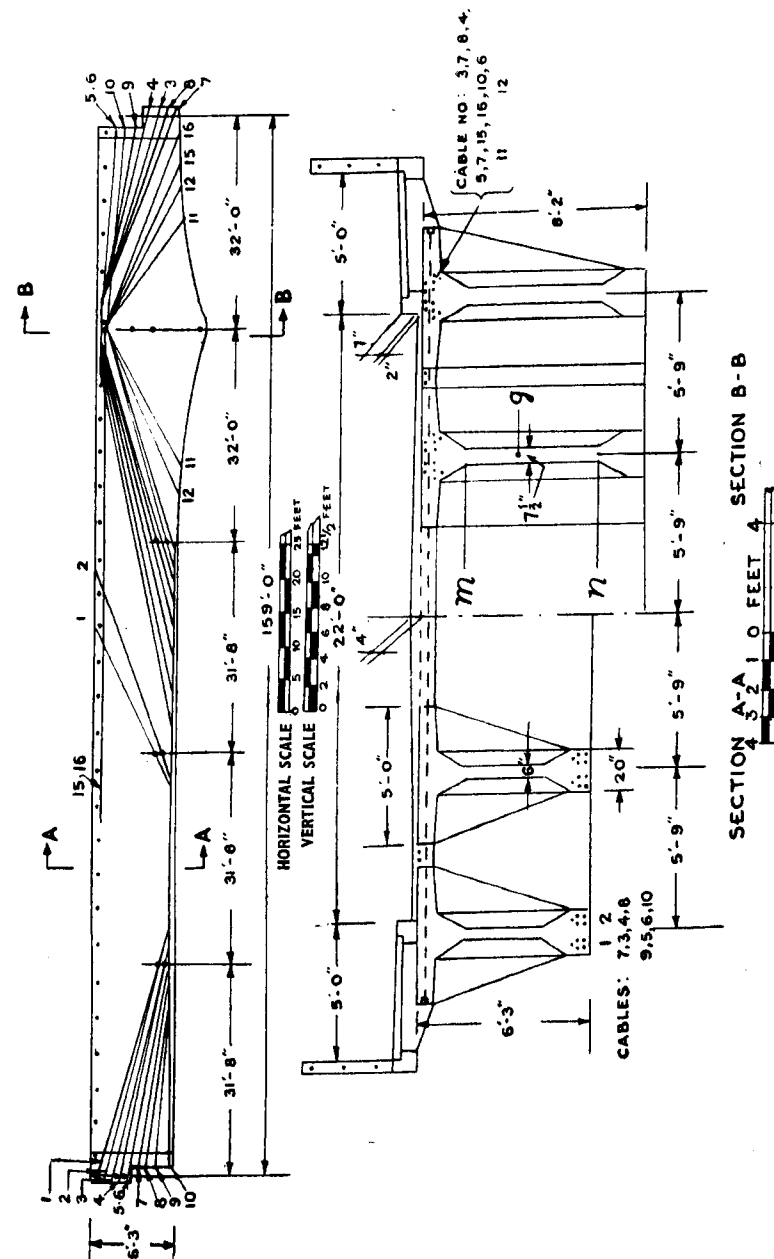


Fig. 9. Cabling of Type II Beams

11.2.6. Each span except the first contains 4 beams and the first span contains 5 beams. The beams of the first span are spaced equally at 5 ft 9 in. centres and the four beams of the succeeding spans are placed in alignment with the four outer beams of the first span. Fig. 9 shows a typical cross section of a type II span. Thus the spacing of these beams leaves a clear gap of 6 ft 6 in. for the launching of beams of successive spans. Five beams are necessary in the first span because of its greater simply supported length. A heavier section that would be necessary if the number of beams were to be four was ruled out from aesthetic consideration and the need to simplify and conserve formwork. As the central beam of the first span will block the launching corridor, it is placed in position only after the girders of the 14th span are launched. Thus it is the 57th and last beam to be launched.

11.2.7. The prestressing cables made up of 12 numbers of 7-mm diameter high tensile steel wires along with the special sheath are placed in the forms of the beams before concreting. As soon as the concrete has acquired sufficient strength, the cables are tensioned and grouted.

11.2.8. After all the 57 beams are launched, the transverse cables in the cross beams are threaded through conduits left for the purpose in the diaphragms, and the cross beams are concreted and prestressed. The transverse cables for the deck slab are threaded through conduits in the top flange of the beams; the top flange of the beam along with the cast-in-situ slabs closing the gap between the beams, forms the deck slab. The in-situ slabs of the deck are concreted after the longitudinal cables for them are also placed in position. After the concrete has hardened the deck is transversely and longitudinally prestressed, thus making each span a completely integrated unit. These operations must proceed backwards from the 14th span, since it is essential that each span is subjected to the maximum negative moment due to dead load reactions on the cantilever before the deck slab gets its full dead load and is prestressed.

11.2.9. The Freyssinet system of prestressing is adopted.

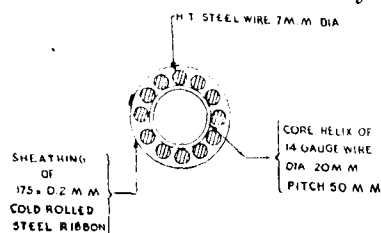


Fig. 10. Freyssinet Cable.

In this the wires of the tensioning cables are held in position round a central open spring (core helix) of 12 or 14 gauge wire of about 85,000 lb per sq. in. ultimate strength, Fig. 10. The bond between concrete and cables which are placed prior to concreting is prevented by covering the

cables with a sheathing made of cold rolled steel ribbon 175 mm by 0.2 mm. The sheathing is made in a special machine. The cables are tensioned by means of Freyssinet jacks and anchored by means of Freyssinet anchorage cones.

The outer cone, usually embedded in concrete at cable ends, is a concrete cylinder with a conical hole, which is heavily armoured with a spiral reinforcement of high tensile wire and is capable of distributing its load on the concrete. The inner cone is made of cement mortar cast round a steel tube and is hooped with wire mesh. The surface of the cone has 12 grooves through which the wires of the cable are gripped. When the inner cone is pressed into the outer cone with the wires threaded between them, concrete is supposed to flow plastically and lock the anchorage gripping the wires firmly.

11.2.10. Over the piers each beam is supported on a separate heavily reinforced precast bed block, Photos 7 and 8. At the time of setting the beam in position a thin layer of mortar is spread over the bed block. For the hinge at the end of each cantilever, which is also the expansion joint, there is a simple arrangement of synthetic rubber plates (Neoprane) alternated with wire mesh to make up a minimum thickness of about 1.25 in., Fig. 11.

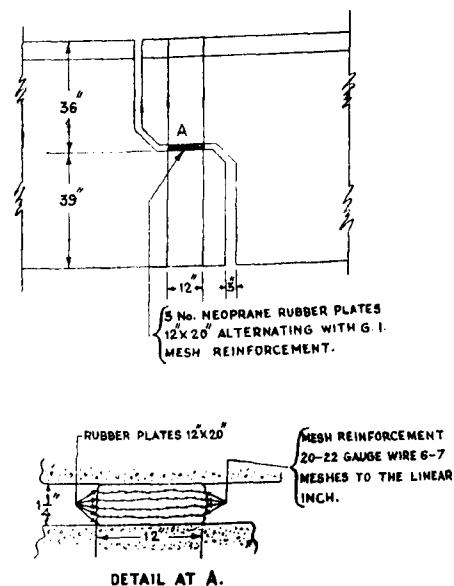


Fig. 11.

11.3. Allowable Stresses

The following working stresses have been allowed in the design of the bridge:

Prestressed Concrete in Service—

Compression: minimum 0, maximum 2,500 lb per sq. in.

Crushing strength on 3 in. cubes at 28 days 7,000 lb per sq. in.

Principal tensile stress 100 lb per sq. in.

High tensile steel—after all losses 1,25,000 lb per sq. in.

11.4. Live Load

The bridge has been designed for the Indian Roads Congress Class A double lane or Class AA single lane loading, whichever produces the worst effect.

11.5. Conditions of Design

11.5.1. The cantilever system adopted in the design of the bridge is statically determinate in service, and in order to ensure that this condition is satisfied during launching also, the points of support during launching correspond approximately with the permanent supports.

11.5.2. As the beams are individually cast, prestressed and launched, during launching each beam carries its own weight, and later, when placed in position carries a portion of the weight of the launching devices and of other beams as they are launched in succession. These subsequent loads are, therefore, live loads which come into operation before the beam has been integrated with the other elements of the span. The dead load on the beam is also increased by the placing of its counterpart in other spans. Therefore, the design of the bridge provides for the following four conditions:

- Each beam acting individually supporting only its own weight.
- Each beam acting individually carrying a portion of the weight of the launching devices and of beams of subsequent spans as moving loads.
- Each beam acting individually supporting the reactions from its counterparts in other spans.
- The bridge in service with full live load, each beam acting as part of the integrated structure of each span.

11.5.3. A combination of conditions (b) and (c) mentioned above is the most disadvantageous of all. Of the other two conditions, the bridge in service leads to the maximum stresses. These are given in Table 3 for one beam of each type. It may be seen that the stresses are all within the permissible limits

TABLE 3
Stresses in Beams under full Live Load

Type	Stresses	S_3'	S_1	S_2	S_3	G
I outer beams	F_p	1,320	1,630	1,680	1,570	1,040
	F_p'	430	330	240	10	660
	F_n	590	460	330	290	400
	F_n'	1,610	2,220	2,420	2,080	1,620
II-1	F_p		1,190	1,250	1,400	890
	F_p'		400	240	0	740
	F_n		430	110	140	130
	F_n'		1,790	2,310	2,290	2,010
III	F_p		1,220	1,580	1,300	
	F_p'		630	30	510	
	F_n		720	930	800	
	F_n'		1,560	1,210	1,440	

and the cabling satisfactory. The extreme fibre stresses due to bending moments are figured from the expressions:

$$f = \frac{12000 M}{Z}$$

$$\text{and } f' = \frac{12000 M}{Z'}$$

where f and f' are the stresses in lb per sq. in. in the top and the bottom fibres, M the bending moment in ft-kips, and Z and Z' the moduli of section in inch units. The prestresses developed in the extreme fibres are calculated from the formulae:

$$p = 1000 \left(\frac{P}{A} + \frac{Pe}{Z} \right)$$

$$\text{and } p' = 1000 \left(\frac{P}{A} - \frac{Pe}{Z'} \right)$$

Note: F_p and F_p' are stresses in top and bottom fibre resulting from dead load plus live load (positive) plus prestress. F_n and F_n' are stresses in top and bottom flange resulting from dead load plus live load (negative) plus prestress.

where p and p' are the prestresses in the top and the bottom fibres in lb per sq. in., P the prestressing force in kips, e the eccentricity of the cables in in., A the area of section in sq. in., and Z and Z' the moduli of section in inch units.

11.6. Shear Stresses

11.6.1. The shear stresses are calculated by the formulae:

$$q = \frac{VN}{Ib}$$

where q is the shear stress, V the maximum shearing force, N the moment of area of the section situated above the given fibre, I the moment of inertia of the section, and b the width of the section at the given fibre. The principal tensile stresses are figured from the usual formula:

$$f_p = \frac{1}{2} \left\{ f \pm \sqrt{f^2 + 4q^2} \right\}$$

where f_p is the principal tensile stress, f the stress due to the bending moment corresponding to the maximum shear, and q the shear stress. Since the critical values are liable to occur at m , n and g , Fig. 9, the principal tensile stresses have been evaluated for these fibres and are tabulated in Table 4. It may be seen that the permissible value is not exceeded.

TABLE 4

Principal Tensile Stresses	
Sections	Stress lb per sq. in.
G_0	3
G_{14}	4
G_{11} (right)	
m	97
g	75
n	25
G_{12} (left)	
m	50
g	75
n	48
S_3' (upper)	50
S_3' (lower)	55

11.6.2. In the case of sections over piers G_1 to G_{13} , the resultant cable has no inclination and, therefore, the modifying influence of prestress is nil. But owing to the rapid variation in the depth of the beam by virtue of the parabolic haunch, the fibre stresses due to bending moments are inclined, and this has a favourable influence over the shear stresses. This favourable effect, being one of the advantages of prestressed beams with a variable depth, has been taken into consideration by working out the shear stresses for a reduced shear —

$$V' = V - \frac{M}{h \cot c}$$

where M is the bending moment corresponding to V , h is the depth of the beam and c the angle between the intrados and extrados at the section considered, Fig. 12. For the sections G_1 to G_{13} the expression reduces to —

$$V' = V - 0.0147 M$$

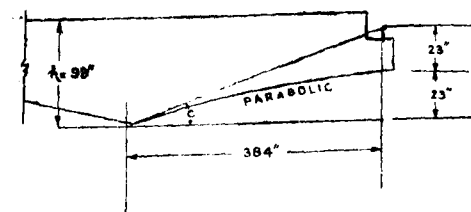


Fig. 12.

The principal stresses given in Table 4, in respect of G_{11} and G_{12} , are based on the modified shear value V' .

11.6.3. In sections at G_0 and G_{14} over the abutments, the bending moment due to loads is nil. So, only the prestress and shear force are considered. The cables have been arranged so that their resultant force passes very near the centre of gravity of the section inducing practically no bending moment. The vertical component of the prestressing force reduces the shear force in the section and the horizontal component induces a uniform compression in the section. The stresses in these sections as set forth in Table 4 have been calculated from these considerations.

11.6.4. To illustrate the treatment of articulations, the case of section S_3' in G_1-G_2 (the second span) is taken. Here the upper end blocks of type II beams rest over the lower end blocks of type I beams transmitting a maximum reaction of 164.3 kips due to dead and live loads at a distance of 15.5 in. from the critical section, Fig 13. In the lower part of the articulation,

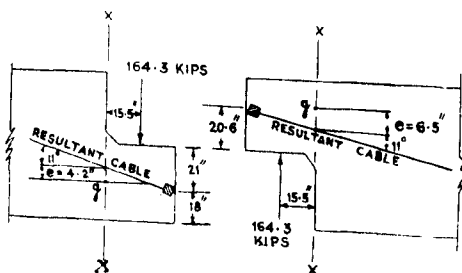


Fig. 13.

this induces a maximum fibre stress of -500 lb per sq. in. at the top and 500 lb per sq. in. at the bottom. The total prestressing force of 450 kips acting along the line of the resultant cable with an eccentricity of 4.2 in. induces fibre stresses of 930 lb per sq. in. and -200 lb per sq. in. at top and bottom. The resultant fibre stresses are 430 lb per sq. in. and 300 lb per sq. in. at top and bottom. The vertical component of the prestressing force reduces the shear force from 164.3 kips to 78.3 kips which produces a maximum shear stress of 150 lb per sq. in. in the plane of the centre of gravity. The principal tensile stress at this point is -55 lb per sq. in. Similarly, the principal tensile stress in the upper part of the articulation is found to be -50 lb per sq. in.

11.7. Cross Beams

The cross beams are designed as prestressed T-beams taking a portion of the slab equal to $1/6$ of the total span of 23 ft on either side as the effective flange. This, of course, does not apply to the cross beam in the lower part of the hinge, which is only of rectangular section. The maximum moments are caused by Class AA loading. The final stresses after algebraically adding the prestresses to the stresses caused by the external moments do not exceed 765 lb per sq. in. in any case.

11.8. Rubber Plate Bearings

11.8.1. The rubber plate bearings in the articulations and over the north abutment fulfill the functions of roller and rocker bearings while ensuring perfect stability under vertical and horizontal forces. By the introduction of these simple bearings, the conventional types of bearings have been dispensed with. They are built up of $\frac{1}{4}$ in. rubber plates alternating with mesh reinforcement which prevents relative sliding between plates, Fig. 11. In service, they attain a state of hydrostatic equilibrium and function by virtue of their ability to undergo distortion to

accommodate temperature strains or end rotation caused by deflections.

11.8.2. Plates of synthetic rubber are used as natural rubber is liable to deteriorate under continued exposure, and have the following mechanical properties:

Shore hardness	50-55
Density	1.35
Ultimate strength	2,130 lb per sq. in.
Ultimate elongation	800 per cent.
Elastic modulus	The modulus of compression with mesh reinforcement, E and the modulus of shear G corresponding to a shore hardness of 50 are:
	E 21,300 lb per sq. in.
	G 142 lb per sq. in.

11.8.3 Area of Bearing

Around the edges of the piling of plates constituting the bearing, there is a reduction in the intensity of compressive stress. Again, the end rotation of the bridge alters the state of stress distribution from uniform to trapezoidal in the direction of the span. In order to minimise the effects of these phenomena, the average compressive stress should be limited to 800 lb per sq. in. Since the maximum reaction is 164.3 kips, the minimum area of bearing should be 205 sq. in. The area actually provided is 240 sq. in., the size of the plates being 12 in. by 20 in., with the shorter side placed in the direction of the span.

11.8.4. Thickness of Bearing

The thickness of the bearing is computed so as to allow for longitudinal movements of the deck without causing too much distortion in the rubber. The permissible distortion is expressed by the relationship

$$\frac{A_l}{t} = \frac{1}{2}$$

where A_l is the total longitudinal movement of the deck and t the thickness of the piling. The total longitudinal movement comprises of the irreversible movement due to that part of shrinkage and creep taking place after the placing of beams in their final

position and of the reversible movements due to temperature effects. For the average conditions at this site,

$$A_l \text{ (inches)} = 0.004 L$$

where L is the span in feet. Since a single bearing has to accommodate the movement of a whole span, $L=159$ ft, and $A_l = 0.636$ in. Thus, $t=0.636 \times 2=1.27$ in. The bearing provided is made up of five plates alternating with mesh reinforcement giving a thickness of 1.25 in.

Under an end-rotation, d , of the bridge due to deflection, there is a relaxation of strain amounting to $d \times 12/2$ on the cantilever side. In order to maintain the integrity of the piling, this relaxation should be less than the average strain:

$$6 d < \frac{1.25 \times 164.3}{240 \times 21.3}$$

For $d=1/760$ a value which will not be exceeded in service

$$\frac{6}{760} < .04$$

which is quite satisfactory. Of course, to be precise, we should only take the reduced reaction corresponding to the maximum deflection which yields an end rotation of $1/760$, but such refinements are meaningless in view of the very large margin in the inequality stated above.

11.9. Stability Under Horizontal Forces

The tractive effort or breaking force per span is $5 + 0.12 L$ tons = 20.2 tons or 45 kips. Thus the force per beam is 11.25 kips. This force is kept in equilibrium by the friction at the "fixed" bearing of precast bed block over the piers.

12. MATERIALS

12.1. Concrete

12.1.1. To obtain the specified strength for the concrete consistently and economically, the mix has been designed on a rational basis taking into consideration the conditions which were, by no means, ideal at the site. The best available coarse aggregate, which had to be transported from a distance of 70 miles by rail, is only of fair quality and has a Stewart impact value of 18 per cent. The sand is rather fine having a fineness modulus of only 2.09. The thin web of the beams and the crowding of cables tend to aggravate the difficulties of concreting,

but these difficulties are, to a great extent, offset by the use of steel side forms and powerful form vibrators.

12.1.2. The water-cement ratio—strength relationship was first determined. The mean strength for which the concrete must be designed was then found out taking into account the standard deviation.

For establishing the water-cement ratio—strength relationship, a number of mixes with water-cement ratios ranging from 0.32 to 0.45, ratios of sand to coarse aggregate ranging from 30–70 to 45–55, and limits of workability required for the conditions of the work, were designed. Some 300 cubes of the various mixes were tested for 7 and 28-day strengths. From a statistical analysis of the test results, the maximum value of the standard deviation was found to be 730 lb per sq. in. Applying this to the probability curve representing the dispersion of test results, the mean strength for which the mix must be designed to yield a minimum strength of 7,000 lb per sq. in. at 28 days was calculated. The corresponding water-cement ratio was found to be 0.35 and the theoretical mix, as finally designed, was 1: 1.26: 2.38 by weight.

12.2. Modulus of Elasticity

In view of the importance of strains in concrete during and after prestressing, the modulus of elasticity has been determined on a number of work test cubes. During the compression tests, the strains were measured by means of a Baldwin strain-measuring instrument using SR-4 electrical strain gauges. Typical stress-strain curves are shown in Fig. 14. It may be seen that at 90 days the concrete has a modulus of elasticity of 6.4×10^6 lb per sq. in., and at the age of 7 days, when prestressing is done, the value is about 4.4×10^6 lb per sq. in. The later value is about 70 per cent of the modulus obtained by tests on 90 days old cubes and may be considered satisfactory.

12.3. High Tensile Steel

The high tensile steel wire for the work is of 7 mm diameter with an ultimate strength of 95–110 tons per sq. in. and with 0.1 per cent proof stress of more than 80 tons per sq. in. Fig. 15 shows a typical stress-strain curve for the steel.

Although cold bending test has not been prescribed as one of the tests for acceptance of steel, as a check on the ductility of the steel, several specimens have been subjected to cold bending tests by bending the specimens once through 180 degrees with a minimum radius of 28 mm (4 times diameter), and all the specimens have successfully withstood the test.

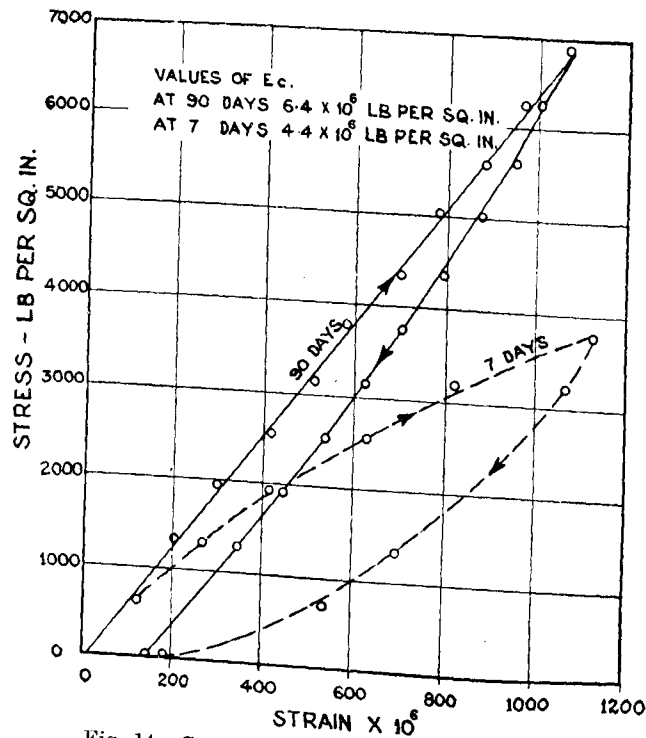


Fig. 14. Stress Strain Curve for Concrete.

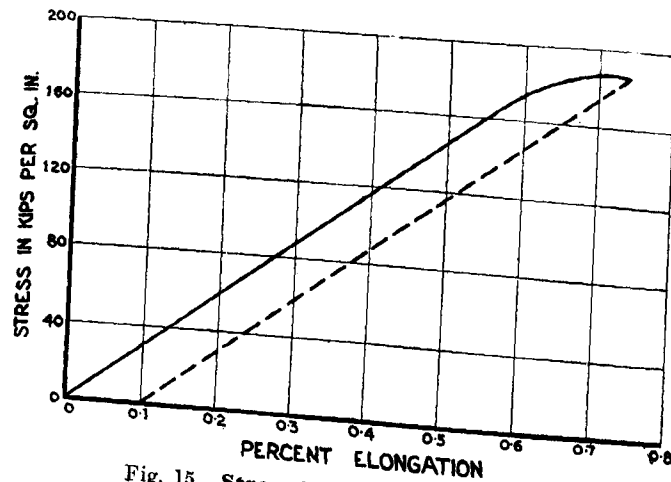


Fig. 15. Stress Strain Curve for Steel.

13. PRODUCTION YARD

13.1. The layout of the production yard has been planned to suit the following sequence of construction operations:

- (a) precasting of beams,
- (b) tensioning of cables and grouting,
- (c) launching of beams,
- (d) concreting of cross beams and deck slab including wearing coat,
- (e) tensioning of transverse cables and longitudinal cables in deck slab, and
- (f) construction of footpaths, handrails, and other ancillaries to complete the superstructure.

The production yard was located on the north approach which had been formed to a sufficiently high level. It was widened to accommodate the various units of the yard. The layout of the yard is given in Fig. 16, page 194. It consists of the cable assembling yard, the casting yards, the stockyards and systems of rail track.

13.2. Cable Assembling Yard

13.2.1. This consists of 12 feeder wheels of high tensile steel wires each about 6 ft diameter arranged in two symmetrical rows, a series of guides and a long bench about 200 ft long, Photos 9 and 10. The wires from the wheels pass through the guides to converge on a core of helix to form the Freyssinet cable. The cable is pulled to the required length by means of a winch connected to the end of the cable by means of a cable grip.

13.2.2. At the other end of the table is the sheath making machine by means of which cold rolled strips 175 mm by 0.2 mm are formed into sheaths for the cables. In this machine the ribbon of thin steel forms a roll which passes through a roller press. The roller press produces transverse corrugations on the steel. Then it passes through guides which form the ribbon into tubular shape and cause a ream of two folds which is practically watertight. Here also a winch is used to pull the sheath.

13.3. Casting Yards

13.3.1. The casting yards consist of two concreting beds, supported on a series of masonry pillars spaced at $4\frac{1}{2}$ ft centre to centre. Each pillar carries a pair of timber sleepers $8\frac{3}{4}$ in.

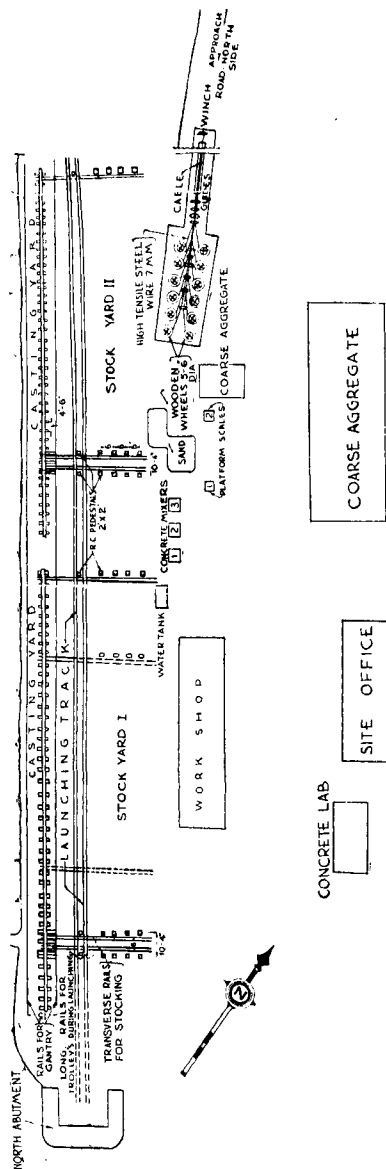


Fig. 16. Layout of casting yard

deep $\times 3$ in. wide $\times 4$ ft 2 in. long, kept together with a clear space of 1 in. in between and anchored down by means of two $\frac{3}{4}$ in. holding down bolts with large plate washer, Photo 11. A circular rubber washer 4 in. diameter and 1 in. thick, sandwiched

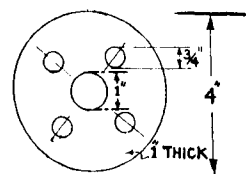


Fig. 17.

Rubber washer for use under casting bed

between 6-in. square plate washers, is interposed between the sleepers and the pier, one at each end. The rubber washer, Fig 17, provides a sort of "floating" support which permits of free vibration of the formwork and absorbs and prevents the vibrations being transmitted to the pillars. Observations during concreting have proved the effectiveness of these washers, for no vibration has been felt on the piers while the full effect has been felt on the formwork.

13.3.2. Casting yard I is designed for the casting of all three types of beams while casting yard II is for type II beams only. With slight alteration yard II can be used for type III beams also. Work in the two yards is so arranged as to take full advantage of the only set of forms available for casting the beams. Thus when concreting is done in casting yard I, there will be a beam just prestressed and ready to be moved out in casting yard II.

13.4. Formwork for Beams

13.4.1. There are as many as 13 sub-types of beams, but the formwork for all the beams is derived from one set suitable for type I with the addition of special sections for the end blocks and diaphragms. Corresponding to the 13 sub-types of beams there are 26 sub-types of end blocks. A general rear view of side forms assembled for a beam is shown in Photo 12.

13.4.2. The formwork is made of mild steel plates $\frac{5}{32}$ in. thick, cut and welded to shape and stiffened by $5\frac{1}{2}$ in. deep webs of the same plate. A good finish has been obtained for the surface and junctions. The formwork is practically free from leakage.

13.4.3. To the back of each section are welded four plates, two of them just above the bottom flange and the other two just below the top flange; each plate has four slots for attaching the vibrators, Photo 12. In practice, it is found sufficient to use only the upper row of fixtures.

To the bottom of each section, four eyelets are welded through which tie bolts are passed for effecting a tight joint between the side and bottom forms.

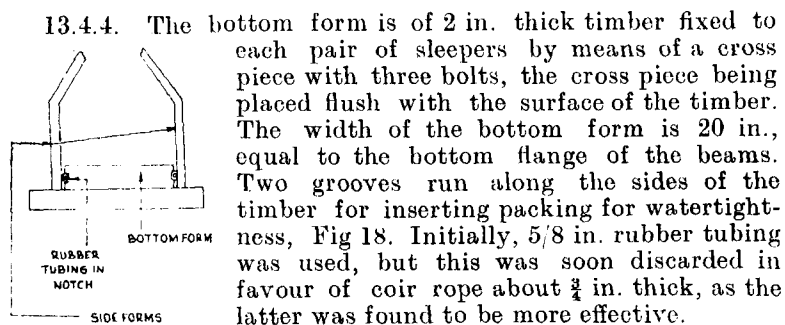


Fig. 18.

Rubber tube packing between side and bottom forms

13.4.4. The bottom form is of 2 in. thick timber fixed to each pair of sleepers by means of a cross piece with three bolts, the cross piece being placed flush with the surface of the timber. The width of the bottom form is 20 in., equal to the bottom flange of the beams. Two grooves run along the sides of the timber for inserting packing for watertightness, Fig 18. Initially, 5/8 in. rubber tubing was used, but this was soon discarded in favour of coir rope about 1/4 in. thick, as the latter was found to be more effective.

13.4.5. Portions of the bottom forms just above the transverse tracks are made of detachable sections which can be removed for jacking up the beam and drawing in the trollies for moving out the beam, Photo 11. The jack for lifting is placed on a ledge of the adjoining pier.

Wherever cables project below the form, detachable sections of short length are provided to facilitate the moving out of the beam from the bed without obstruction from the projecting of cables.

13.5. Gantry

A gantry working on a track of 18-lb rails is used for moving and placing side forms, Photo 13.

13.6. Stock Yards

The two stock yards for storing beams have pedestals of reinforced concrete with suitable foundations for supporting the beams. Each pedestal has a ledge for placing the jack for lifting beams.

13.7. Rail Tracks

13.7.1. There are three systems of tracks: the launching track running down the length of the production yard and ending at the "trans-shipping yard" intended for launching beams; and the two systems of transverse tracks connecting the casting yards with the stockyards and cutting across the launching track.

13.7.2. Trollies for conveying beams along these tracks are illustrated in Photo 14. The cross beam connecting the two units has a segmental bearing on each unit. For moving the beams over the transverse tracks, only a single trolley at the

rear end of the beam, is necessary. Initially, there were only two sets of trollies. As these were found insufficient and delays were often experienced in moving beams, two more sets incorporating certain improvements were built. Photo 15 shows one set with the cross beam removed to expose the segmental bearing.

14. PRODUCTION BEAMS

14.1. The sequence of operations in the casting of a beam is as follows:

- precasting end blocks and placing them in position,
- erecting western side forms and placing western half of the nominal reinforcement,
- assembling and placing high tensile steel cables and lining them up to correct profile,
- placing eastern half of nominal reinforcement and erecting eastern side forms, and
- concreting.

14.2. Precasting End Blocks

14.2.1. The end blocks are heavily reinforced in order to be able to take up and distribute the stresses developed in the anchorage zone.

After one side form (usually western) and end pieces are erected, the outer cones are fitted. The standard fixture for the outer cone consists of two small studs screwed into the external face of the cone through the steel form. Finding this inadequate to keep the anchorage firmly fixed during concreting, two more hook bolts have been provided as shown in Fig. 19.

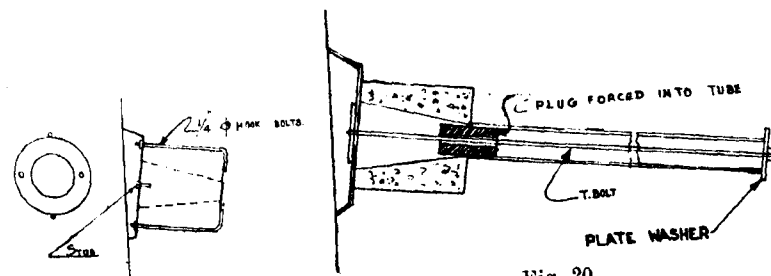


Fig. 19.

Fig. 20.

14.2.2. The cable conduits are formed by inserting 1 1/2 in. bore gas pipes which are secured to the cones by means of a long

T-bolt as shown in Fig. 20. Where anchorage cones are fitted with a collar, the plug shown in the figure is not necessary as the pipe can be inserted over the collar straightaway. The junction of pipe and cone is sealed with putty or mastic to prevent the escape of mortar into the cone. It is best to remove the mortar that might have got deposited in the cone after concreting.

14.2.3. In placing the mild steel reinforcement, it is best to begin with the bars nearest the cone and work towards the other end. Thus the spirals round the cones are first placed, then the first row of grills, and so on. Photos 16 to 18 show some details mentioned above.

14.2.4. After every detail is fitted into the appropriate place, the open side of the form is closed and pipes for the transverse cables conduits inserted.

14.2.5. Three form vibrators are mounted for compacting the concrete: one 3-phase 50-cycle 1,500-watt vibrator at the bottom and a 3-phase 50-cycle 850-watt vibrator on each side. This was the original arrangement for all three types of end blocks but owing to the peculiar shape of the upper end block it sometimes led to some honeycombing in the upper part of the articulation. This has been overcome by arranging to fit another 850-watt vibrator at the bottom of the upper part of the articulation. The bottom vibrator is switched on as soon as concreting starts and is worked throughout if required. The side vibrators are switched on only after the concrete has come up to about half the height and are continued till the end. In the upper end blocks, the vibrator at the articulation is switched on after the concrete has just come up above the horizontal ledge and then the bottom vibrator is switched off.

A tricky spot in the lower end block is the lower part of the articulation. In order to make sure that concrete flows into this ledge properly, it is kept open at the top and the closing piece is inserted only after the concrete has filled up to the proper level. This opening is sometimes used for feeding concrete also in the initial stages.

14.2.6. During concreting, the fixtures carrying the anchorage cones are frequently checked to ensure that they are in correct position. Before rotating the T-bolt is removed, the pipes embedded in the concrete to form the cable conduits are turned round and round after an hour to prevent them from being gripped by the concrete as it sets, and they are pulled out after about 3 hours.

14.2.7. After the forms are erected, it takes an experienced fitter with two assistants about 6 hours to complete all the fittings and get the forms for the two end blocks of a beam ready for concreting. The concreting takes about one hour. The forms are stripped after a lapse of about 12 hours, and curing with water spray starts immediately.

In the spare space available in the casting beds end blocks are concreted in advance and kept in readiness for use when required.

14.3. Assembling Freyssinet Cables

14.3.1. The high tensile wires of 7 mm diameter are received in rolls of 6 ft diameter weighing on an average 350 lb. On the site the wires are stored in a dry place to prevent them from rusting.

Twelve rolls of wire are placed on the feeder wheels of the cable yard and the wires are threaded through the various guides. As the wire is self-straightening, care should be exercised in opening the rolls to avoid injury to workmen by being lashed by the wire.

The core helix of 14 gauge wire is made at the site by spinning a close spring of 26 mm diameter and stretching it about 25 times to give a helix 20 mm diameter and 50 mm pitch. A large quantity of this helix is wound on a drum from which it is fed into the cable through the central opening of an intermediate guide where the wires get arranged in a circle 6 in. in diameter.

14.3.2. The cable is pulled over the long table. As it emerges out of the guides, oiled and in proper shape, it is tied with binding wire at about 3 ft intervals. When the specified length is reached, the wires are nipped off by means of the special cutter provided for the purpose. The cable lengths are calculated allowing an extra of 2 ft from the face of the anchorage cone at each end for mounting the prestressing jack. For ready reference, the various cable lengths are marked on the long bench.

The core helix which runs the full length of the cable is cut at a distance of about 6 ft from either end of the cable. These two bits are left in position until the prestressing operations commence.

14.3.3. The cold rolled strips, 175 mm x 0.2 mm, for making the sheaths are received in rolls weighing about 315 lb and contain about 1,700 ft of strip. When the appropriate length is reached, the sheath is cut by hack saw. The sheath holder is removed from the cut piece and refixed on the machine side to continue sheath making.

The bench can accommodate some six cables and their sheaths side by side. Hence after six cables are formed, the winch is connected up for pulling the sheath.

The sheaths are made shorter than the respective cables by 4 ft plus the length of cable traversing the end blocks in the case of cables anchored in them. In the case of cables anchored in the top or bottom flange of the beam, the sheath is made shorter by about $2\frac{1}{2}$ ft for each anchorage to allow for the length of the cable from the back of the anchorage cone.

14.3.4. To insert a cable inside the sheath it is carried back from the bench by a number of men standing at intervals of about 5 ft along the length of the cable. It is then slowly brought forward and guided into the sheath which is nailed down near the point of entrance. In the earlier stages of the work, a pilot rope and cable puller used to be employed for the purpose but were soon discarded having been found unnecessary. The finished cables are kept under cover until they are required for use. Usually, the cables are assembled only a short time before they are placed in the beams. It takes from one to two days to assemble the 14 cables required for a beam.

14.4. Erection of Side Forms

14.4.1. After the profile of the soffit is checked, the end blocks are moved up to position by means of the gantry, lifting each clear of the bed by two hooks embedded in the top. The weight of each end block being about 2.7 tons, two differential pulleys of $1\frac{1}{2}$ ton capacity each are used for lifting.

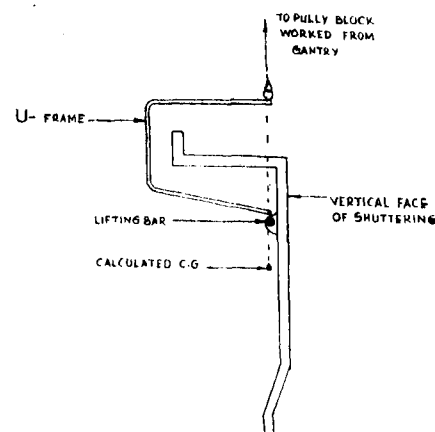


Fig. 21. Device for lifting shuttering

14.4.2. The side forms, generally the western side, are next erected section by section beginning with the diaphragm shutter at *G* and working both ways. For picking up the shuttering a U-frame, Fig. 21, is used. Photo 19 shows the western side shutters erected for a beam.

In order to ensure a slippery non-adherent finish, the forms are smeared with crude oil in which washing soap has been melted at the rate of $1\frac{1}{2}$ bars per gallon.

14.5. Placing Cables in Position

14.5.1. After the western side forms are erected, the nominal mild steel reinforcement is placed in position. The mild steel in the top flange, the vertical bars and the longitudinal bars to go on the western side are welded into cages of about 40 ft length and placed in position. The cable hangers are suspended from the appropriate transverse pipes.

The cables are carried, by one man at every 5 ft in order to avoid sharp sinuous sagging of the cable which may crack the sheath, and placed in the form commencing with the lowest cable in the upper end block and working upwards, the appropriate level in the hanger being used. On reaching the lower end block all the binding wire knots in the exposed portion of the cable are removed, retaining only one knot at the very tip. This precaution is intended to prevent any binding wire slipping down and fouling the anchorage. The cable is then pushed through the lower end block and the end clamped in position by a cable securer. The cable securer consists of a flanged tube, a wedge, and a clamp as shown in Photo 20 and Fig. 22.

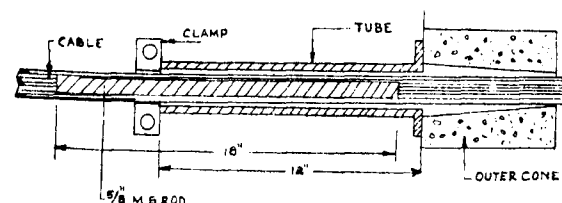


Fig. 22. Cable securer as assembled

14.5.2. The other end of the cable is then inserted through the upper end block after taking out the portion of the cable between the first hanger and the upper end block in a wide sweep bringing the cable tip within the end block.

After placing all the cables in position, the balance of mild steel reinforcement is placed. The cables are then lined up to the correct profile providing additional supports, if necessary, and the cable ends at the upper end block are also clamped by cable securers.

14.5.3. When cables are anchored in the top or bottom flange, the anchorage cones are fitted to cone boxes, Photo 21, made to suit the angle of the cable. These cone boxes are made of steel and can be used repeatedly.

14.5.4. In the case of cables which are anchored at one end by bond, for example, No. 15 and 16 in type II

beams the wires are fanned out, Photo 22, for a length of 10 ft. Where the cable emerges out of the sheath, a spout is fitted for grout to flow out during the grouting of the cable and well caulked with cement mortar. The side forms on the other side are erected and checked. It is important to loosen the tie bolts in the web about an hour after the concrete has passed a particular bolt and rotate it frequently to avoid seizing. These bolts are taken out after two hours.

14.5.5. When the bottom tie bolts are tightened, the side forms press hard against the bottom form and press against the packing of rope making the joint leak proof. All vertical joints are packed with moist clay and ribbons of jute cloth just before concreting.

Among the last details to be attended to are the placing of tubes for forming the transverse cable conduits, and if it is an exterior beam, the fixing of cones with their anchorage zone reinforcement along the outer edge of the top flange. The tube fixture is removed one hour after concreting round it. It is rotated from time to time and pulled out after the lapse of two to three hours.

For interior beams, the same tube is used, but to prevent their being disturbed during vibration, a long rod is pinned through the key holes at the end of the tubes and secured to the channel ties at the top.

The exterior beams carry the bond rods for the cross beams of the footpath, anchored in the web. These rods are placed in position by means of a wooden frame clamped to the top flange of the side forms and detached after concreting. The stirrups for the kerb are stuck in to the concrete when it is still raw.

14.6. Adjustment of Beam Length

It is possible for beams to vary in length up to a maximum of $2\frac{1}{2}$ in. All the beams of any span must match absolutely and some adjustment of length may be necessary on account of small difference in span length. Since the diaphragm at *G* is a fixed reference and the side form units are all fixed, the necessary adjustment of length is more conveniently done by moving the end blocks to give a specified overlap of the side forms on the web of the end blocks. The lower end block, being stable by itself, is easily set to a given mark on the bottom form in the beginning itself, but the upper end block is top heavy and unstable and, therefore, very difficult to be set to a given mark. The final adjustment in the position of upper end block is more easily made after the beam assembly is complete.

These final adjustments in length may lead to or necessitate some displacement of cable securers; they are wedged and clamped again with the flange of the tube bearing firmly against the face of the anchorage.

14.7. Concreting of Beam

14.7.1. The coarse aggregate is screened to remove dust, washed and stacked at least two or three days before it is used so that the stack drains out and attains a fairly uniform state of moisture content. To avoid segregation, the stack is built up limiting the height to 4 ft. To prevent segregation of the different sizes in the aggregate, when the washed aggregate is dumped from baskets, the stack is built in stable ledges, each basket load staying put where it is dropped.

The best available sand is a medium sand and is collected from specified areas of the river bed and stacked in advance of requirements. This advance stacking permits the attainment of uniform moisture content in the sand.

As a matter of routine check, tests for gradation, unit weight, specific gravity, silt content and organic impurities are performed from time to time, but as results are observed to be consistent it has not been found necessary so far to modify the theoretical mix.

The inflow of cement from producers is so regulated that period of storage is a minimum.

14.7.2. The batching is done entirely by weight. It was found that the net weight of cement per bag was 109 lb due to loss through the meshes of the jute bag. Thus, for a batch of one bag of cement, weights of the various materials were:

	lb
Cement (1)	109 (one bag)
Sand (1.26)	137
Coarse aggregate (2.38)	259
Water (W/C ratio 0.35)	38

The weights for the field batch are determined by correcting the above values for the moisture content of the sand and the coarse aggregate. The moisture content is determined by a pycnometer bottle in the morning before starting the concreting. The tests are repeated whenever significant changes occur in the condition of the materials.

14.7.3. Although the concrete is compacted by heavy form vibrators of 1,500 and 850 watts, owing to the slenderness of the beam section and the crowding of cables, it is imperative to ensure the maximum of workability in order to avoid honeycombing. There are certain disadvantages in getting this workability by water alone. Under prolonged vibration the flow of concrete becomes difficult to control and this by itself can lead to a different pattern of honeycombing. Also, the cement content has to be increased to maintain the water-cement ratio at 0.35. This is not only uneconomical but holds possibility of early shrinkage cracks occurring. These drawbacks have been overcome in the design of the mix by keeping the water content to the scale of "no slump" concrete and getting the necessary workability with the aid of Lissapol N. One lb of a stock solution of $\frac{1}{2}$ oz of Lissapol N in one lb of water is added to the mixing water of every batch. The total quantity of water added including the Lissapol N-Solution is calculated for a water-cement ratio of 0.35.

14.7.4. The order followed in loading the hopper is: first, the coarse aggregate, next, the cement, and finally, the sand. The mixing water is first admitted into the drum and then the hopper is emptied into it. The speed of rotation of the mixers is regulated to give the best possible mix. This ranged from 30 to 45 revolutions per minute.

It has also been observed that if the emptying of drum is done too slowly, segregation takes place with the coarser fractions dropping out first, followed by the fines. This is avoided to a large extent by tilting the drum to discharging position in a smart and deft sweep.

Concrete is conveyed to the job by head loads in steel pans.

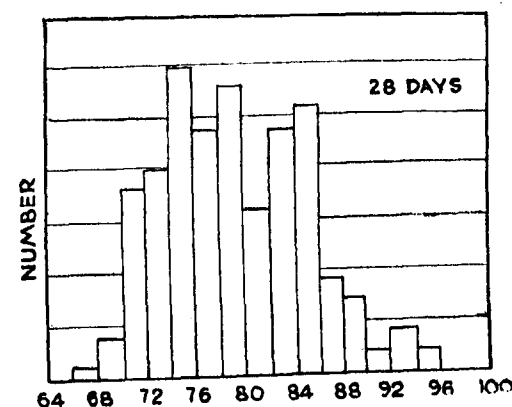
14.7.5. For compaction, there are 10 form vibrators: 4 numbers of 3-phase 50-cycle 1,500-watt capacity for the bottom, and 6 numbers of 3-phase 50-cycle 850-watt capacity for the sides. The vibrators work on 45 volts current obtained through step-down transformers from the 440 mains.

14.7.6. Concreting is done beginning from the upper end block and working towards the lower end block. Other methods such as beginning from the lower end block and working towards the upper end block, beginning from section G and working both ways, have been tried and discarded having been found unsuitable for the lie of the cables. As such, at the commencement of concreting, the 10 vibrators are mounted to successive holders, commencing from the upper end block, four at the bottom and three to each side.

14.7.7. As concreting is done to the full depth of the beam in a continuous operation without permitting any construction joints, the job face advances at an inclination of 30 to 60 degrees. In consequence, the bottom vibrators will be ahead of the side vibrators; normally, one or two bottom vibrators and two vibrators on each side will be working while the others, after doing their work in particular positions, are moved to advanced positions. To be really effective, the vibrator must be fixed tightly to the holders. When a vibrator is properly fixed, the vibrations can only be felt and not seen, since a hazy outline betrays a loose fixture and not effective vibration as may be easily mistaken.

14.7.8. Both in the manner of depositing concrete and in the operation of vibrators, especially those at the bottom, certain precautions have been found advantageous in avoiding segregation. For example, deposit from the bottom upwards in ledges, and not on the slope, and do not switch on the bottom vibrator unless concrete has filled in to about half the depth of the bottom flange. A danger to be constantly guarded against is that of "undermining" resulting from the flow of concrete in the web alone leaving a small tunnel to develop just below the flange. This is liable to happen when a new bottom vibrator is switched on when the job face is too steep, and must be remedied by cutting back the flange to open the tunnel and expose the web slope fully.

14.7.9. The slump cone did not record any slump for the stiff mix used. Wigmore Consistometer gave erratic results.



CRUSHING STRENGTH IN 100-LB PER SQ. INCH

Fig. 23. Distribution of compression strength.

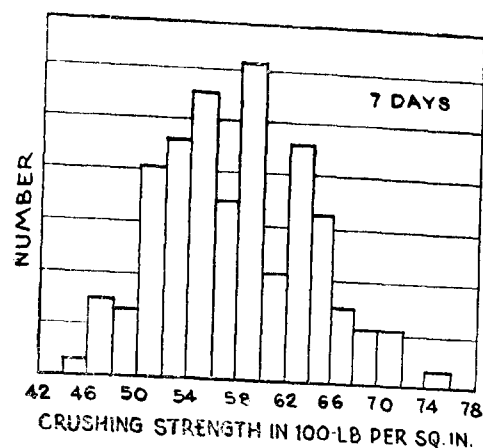


Fig. 24. Distribution of compressive strength

So, the time required to compact 6-in. test cube was adopted as a measure of the consistency.

Once in the morning and once in the afternoon, 5 test cubes are cast for compression tests, the cubes being compacted on a Johnson Vibrating table until the matrix just appears on the surface. With the 5 cubes cast during the concreting of end blocks it makes a total of 15 test cubes per beam. The cubes are tested at 7, 28 and 90 days. The actual distribution of 28-day and 7-day strengths is shown in Fig. 23 and 24. It has not been possible to crush most of the cubes at 90 days in the 150-ton testing machine available on the job.

14.7.10. Some factual data concerning the concreting of a beam are given below. The values are typical.

Beam II₆

	End blocks	Running sections
Concreting time	1 hr. 5 min.	6 hrs. 30 min.
No. of batches used	22.5	376
Solid volume, cu. ft.	78.2	1,309
Volume, planned dimensions, cu. ft.	76.0	1,280
Over run (+) per cent	2.9	2.3

Materials for cubic yard of concrete

Cement	847 lb
Sand	1,065 lb
Coarse aggregate	2,010 lb
Water	207 lb
Lissapol N in solution	3.9 oz

14.7.11. It is imperative to remove the "key" shutters - those at the diaphragms in the night. Otherwise when shrinkage sets in, flanges of side forms are pressed together making it harder to remove the forms as hours pass. Usually, key shutters are removed starting at about 10 o'clock on the night of concreting, and by 12 noon on the following day all the forms are removed.

14.7.12. The beam remains in the casting bed for about a week after which a minimum of 6 cables are prestressed to make the beam self-carrying. The beam is then moved out and set upon a set of storage pedestals where, after about a week, the other cables are prestressed. The wires are then trimmed by oxy-acetylene flame, and the cables grouted with cement mortar. The mix for the grout has the following proportions:

	lb
Cement	109
Fine sand	20
Water	70

The grout is mixed continuously in a colcrete mixer and injected through a nozzle after the cables are flushed with water under pressure, then with compressed air to drive out water that might be trapped in the sags. When clean grout comes out at the other end of the cable the grout holes at both ends are plugged.

Finally, the anchorage recesses are finished off with concrete. Photo 23 shows a wooden form being fitted for concreting.

14.7.13. During concreting of the beam, the top flange exposed to the sun is liable to dry up quickly, and so it is protected against loss of moisture by moist hay. Curing with water spray starts after 24 hours and is continued for 21 days.

15. CALCULATIONS OF STRESSES AND ELONGATIONS IN THE CABLES

15.1. In a prestressed concrete beam, the initial stresses induced in steel by the stressing of cables are reduced in the

course of time due to the following causes :

- (a) Due to the creep of concrete under the permanent compression, ΔT_1 .
- (b) Due to creep of steel, ΔT_2 .
- (c) Due to other causes, the most important of which are : part of the shrinkage of concrete which takes place after prestressing ; relaxation of a cable due to prestressing of other cables ; and inward movement of inner cone in the process of locking the anchorage which is significant only for short cables.

15.2. Besides, stresses vary along the length of the cable due to friction between the wires and the metal sheathing. The friction depends upon the length and curvature of the cables. The stress in a cable is maximum near the anchorage zone since the prestressing force is developed at the anchorage through the external application of the jack.

15.3. Required initial tension at critical sections is calculated as follows :

Loss due to creep of concrete under permanent compression—

$$\text{For cables not injected, } \Delta T'_1 = \frac{E_s}{E_c} \times \frac{F}{S}$$

where E_s = modulus of elasticity of steel

E_c = modulus of elasticity of concrete for deferred strains

F = total prestressing force

S = area of concrete section

Thus F/S = prestress in neutral axis of section.

A value of 9 is satisfactory for E_s/E_c .

For cables bonded by grouting

$$\Delta T''_1 = \frac{E_s}{E_c} \times \text{permanent compression in concrete at the level of the resultant cable.}$$

As the cables are grouted only some time after tensioning, the actual loss is taken as the average of the two values. Thus—

$$\Delta T_1 = \frac{1}{2} (\Delta T'_1 + \Delta T''_1)$$

Loss due to creep of steel—

For the steel used on this work and the conditions of its usage, the loss is about 5 kg per sq. mm. Thus

$$\Delta T_2 = 5 \text{ kg per sq. mm.}$$



Photo 7. Reinforcement for the bed block

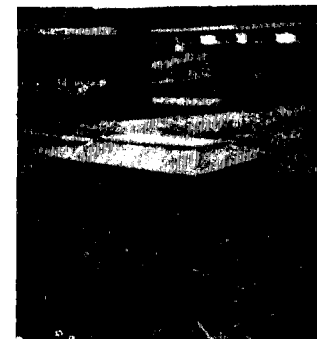


Photo 8. Bed block



Photo 9. Cable assembling yard



Photo 10. Girders in the assembling yard

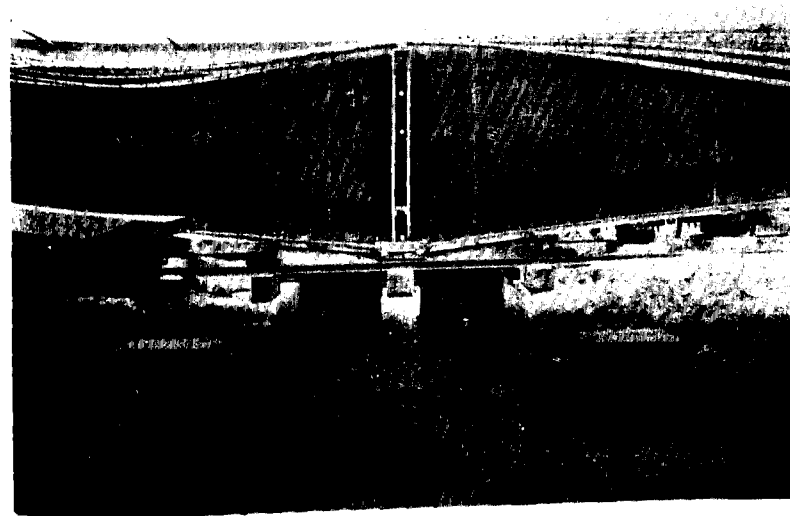


Photo 11. Arrangements for the soffit of the beam

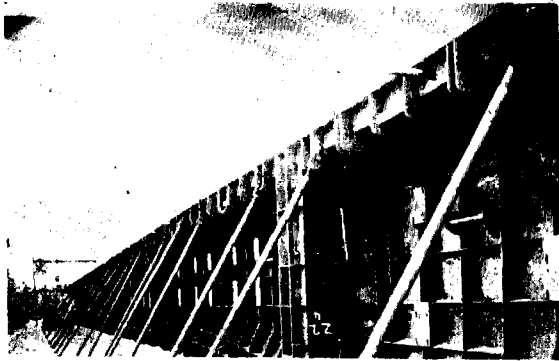


Photo 12. Shuttering for type I beam

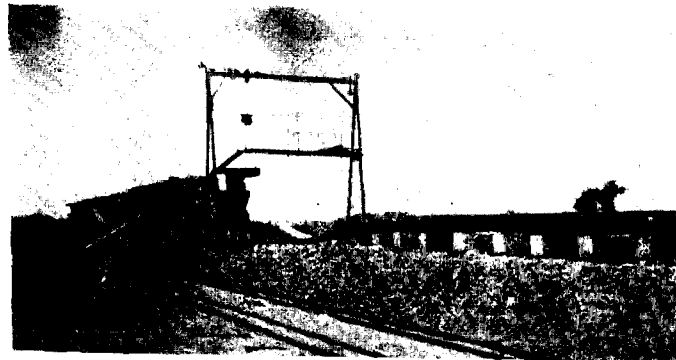


Photo 13. Gantry in the casting yard



Photo 14. Trollies for conveying beams



Photo 15. Locally built trollies for conveying beams



Photo 16. Reinforcement near the cone



Photo 17. Reinforcement for lower end block

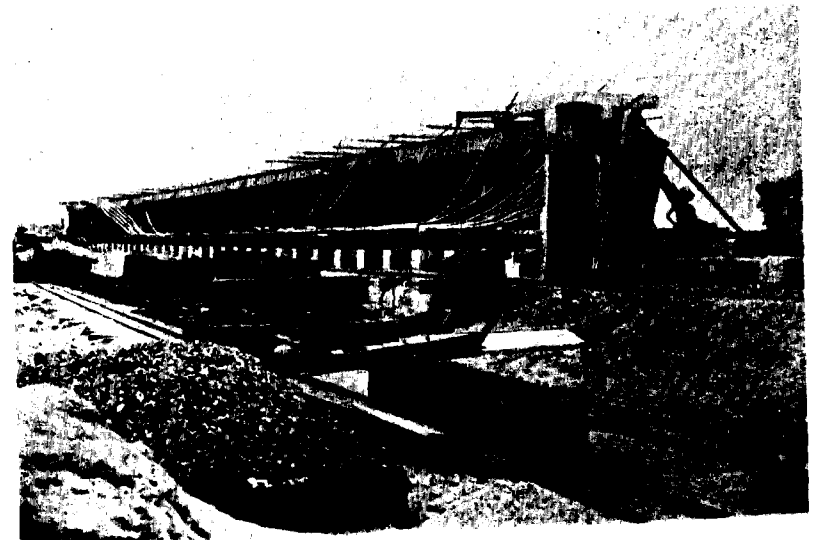


Photo 19. Shutter for one side in position



Photo 18. Reinforcement for upper end block

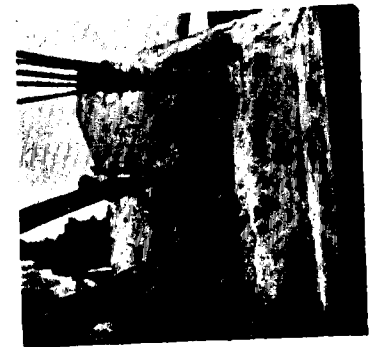


Photo 20. Cables with the cable securer in position

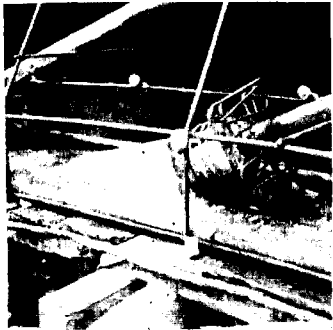


Photo 21.
Cone boxes

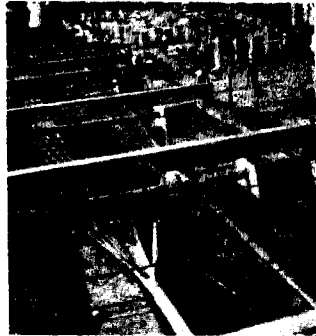


Photo 22. Fanning out of
wires in Cables No. 15 and 16



Photo 27. Girders being moved
by transport cars in span 2

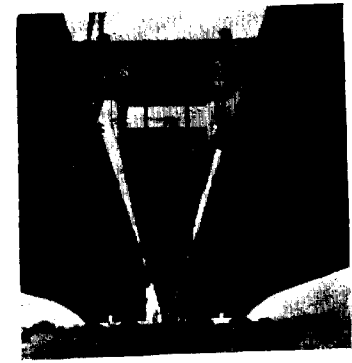


Photo 28. A beam being moved
over girders laid in position



Photo 23. Wooden form for con-
creting the anchorage recesses

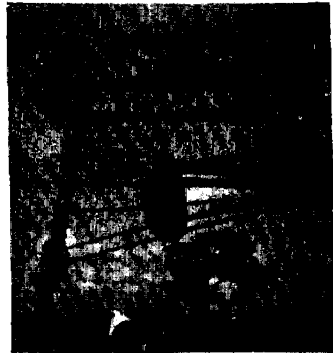


Photo 24.
Spreader disc

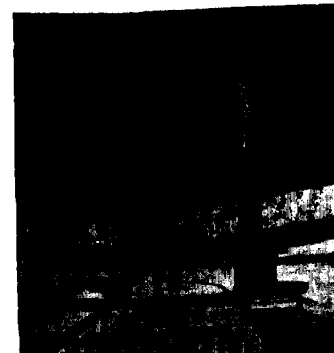


Photo 29.
Shifting box over roller grids



Photo 30.
Tail piece



Photo 25. Prestressing Jack
in position for stressing



Photo 26. Wires are
fixed by wedges

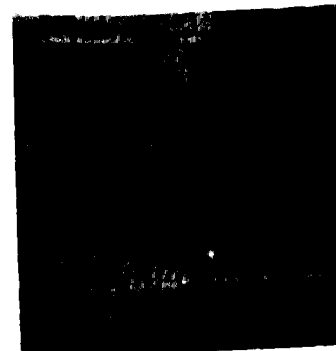


Photo 31.
Articulation



Photo 32. Erection of
launching girder

Loss due to other causes—

Loss due to shrinkage of concrete under normal conditions of work = 3 kg per sq. mm. Loss due to prestressing of other cables = 1 kg per sq. mm.

Design stress after all losses = 88 kg per sq. mm.

Therefore, the required initial tension at critical sections
 $T_i = 88 + \Delta T_i + 5 + 3 + 1 = 97 + \Delta T_i$ kg per sq. mm.

15.4. Tension at ends of cables

The tension exerted at the ends of cables diminishes along the length of the cable owing to friction. The total loss is composed of—

- (a) loss in anchorage cones,
- (b) loss in straight portions, and
- (c) loss in curves.

Loss due to (a) has a constant value of 4 per cent.

Losses due to (b) and (c) are given by

$$T = T_o \left[1 - \frac{0.4 (l d)}{100} \right]$$

where T = tension at any point of cable

T_o = tension at end of cable

l = length under consideration in metres

d = cable curvature in degrees.

Average tension $T_m = \frac{2 T_{min.} + T_{max.}}{3}$ for parabolic cables.

Since the cross section area of cable 12/7 mm is 461 sq. mm.
 total pressure in jack = $T_o \times 461$ kg.

Since the area of the tensioning piston of the jack is 160.6 sq. cm., the manometer pressure corresponding to the jack force stated above

$$= \frac{T_o \times 461}{160.6} \text{ kg per sq. cm.}$$

15.5. Reference Table for Prestressing

A complete reference table for prestressing is prepared for each type of beam. Table 5 shows the details for beam II_{I-II}. Typical calculations for cable No. 3 are given in Appendix 1.



Photo 33.
Travelling crab



Photo 34. Launching girder
moving over girders
already laid in position

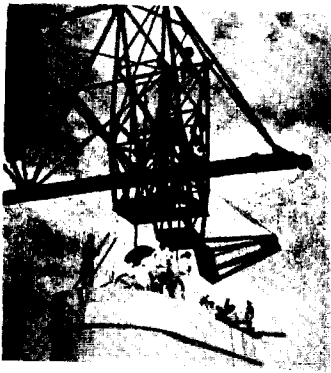


Photo 35. Hinged portion of
the standard being lowered

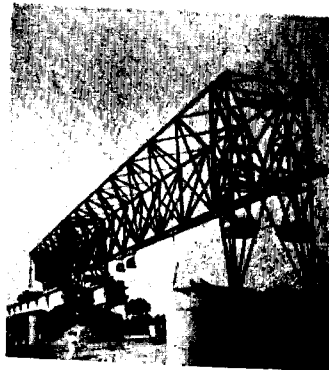


Photo 36. Launching girder
in working position



Photo 37. A beam being
launched by the crabs
and transport car II



Photo 38. Two beams of
one span in position

The order of prestressing given in the Table 5 has been fixed with the following objects:

- (a) To make the beam self-carrying with the tensioning of six cables so that it may be moved out of the casting bed with the least possible delay to enable shutters for the next beam to be placed.
- (b) To distribute the prestressing forces as evenly as possible near the anchorage zones and ends of beam.
- (c) To avoid any significant downward deflection of the cantilever when the beam is still in the casting bed.

TABLE 5
Reference Table for Prestressing

Beam Type II_{I-II}

Cable No.	Order of tensioning	Strain per 1000	Total strain mm	Manometer pressure kg per sq. cm.
1	9	5.45	145	366
2	10	5.45	157	366
3	3	5.10	255	353
4	5	5.10	255	353
5	2	5.15	253	351
6	4	5.15	253	351
7	1	5.00	246	346
8	6	4.95	245	340
9	7	4.95	244	337
10	8	4.95	244	331
11	11	5.00	83	322
12	12	5.00	88	322
13	15	5.40	242	368
14	16	5.25	242	354
15	13	5.05	134	351
16	14	5.10	137	351

16. PRESTRESSING PROCEDURE AND PROBLEMS

16.1. Equipment for Prestressing

The equipment and accessories used for prestressing operations are given in Appendix 2-(d).

16.2. **Freyssinet Jacks**—1951 model, which is now obsolete, was used, Fig. 25, page 218. A description of the jack is given in Appendix 3.

16.3. Labour and Supervision

The following staff was trained in the prestressing work:

For supervision —	No.
Junior engineer	1
Inspectors, for reading elongations, deflections, etc.	3
2 Squads each consisting of:	
Mate—literate, to regulate pumping, read off pressures	1
Fitters, for mounting jack, dewatering, etc.	2
Mazdoors, for operating crane to hoist jack	2
Mazdoors for pumping	3
Mazdoor, literate, for general help	1

16.4. Preliminary Arrangements

Deflectometers for measuring the deflection of the beam during prestressing are placed in position. Bamboo screens and red flags are placed at about 10 ft behind each jack to warn persons, against approaching the jacks from behind since a loose or worn out wedge grip can cause accidents. It was considered desirable to collect data which may lead to a fuller understanding of the properties and performance of prestressed concrete. Further, it has been necessary to ensure that the cantilever did not by any chance suffer a downward deflection so long as the beam was in the bed. Therefore, deflections during prestressing have been measured at important points in the beam.

16.5. Prestressing Procedure

The following sequence of operations is observed during prestressing of beams:

- (a) The cable securer at each end of cable is removed.

- (b) The self-centering collar consisting of a split ring with an external sleeve is placed on the cable in close contact with the anchorage cone.
- (c) The spreader disc is inserted to spread out the wires without twisting or crossing, Photo 24.
- (d) The cut length of helix at each end of the cable is pulled out and it is checked that the end of the main helix in the cable is about 2 ft from the face of the cone. This is important to prevent interference with the locking of the anchorage.
- (e) The inner cone is inserted as far as it will go, with the 12 wires running along the 12 grooves round the cone. The spreader disc is now removed and the cone forced in by light strokes transmitted through a wooden strut held against its back.
- (f) The prestressing jack for each end is hoisted by means of the crane which is secured to the top flange and it is pushed forward coaxial with the cable and with the wires passing through the grooved foot, Photo 25. Here it is important to push the jack as close as possible to the cone face in order that as little as possible of the travel of the jack is lost in initial pumping.
- (g) The wires are placed two by two between the splines and secured by wedges which are driven in by a hammer, Photo 26.
- (h) The tensioning chamber of the jack is connected up to the pump through inlet *D*, Fig. 25.

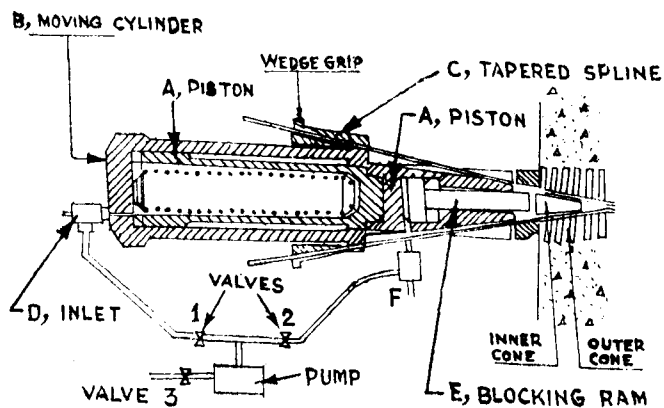


Fig. 25. Freyssinet Jack 1951 model

- (i) Keeping valves 1 and 2 closed and adjusting the pump to operate on the large piston, valve 3 is opened and a few strokes are made till water flows freely out.
- (j) Opening valve 1, keeping 2 and 3 closed, and adjusting pump to operate on small piston, the jack is pumped till the foot of the jack slides into position inside the collar.
- (k) The jacks are pumped at a steady pace till the monometers read 50 kg pressure. The kinks, if any, in individual wires are now eliminated, and the cable is in low tension by virtue of which the jacks have become self-carrying. The hoisting tackle may be slackened sufficiently to permit free movement of tensioning cylinders of the jacks.
- (l) The millimetre scale, Fig. 26, is held against one of the wires in a convenient position butting the dummy end against the cone face, and a crayon mark is made on the wire against the zero of the scale. When the cable is being tensioned, the reading of this mark on the scale gives the elongation in millimetres. Crayon marks are also made on all the 12 wires close to the wedge grips. Any movement of a wire relative to the grips due to slipping is readily indicated by these marks.

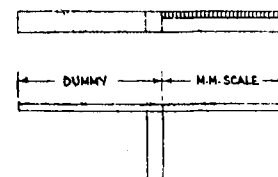


Fig. 26.

- (m) All the wedges are again hammered to ensure their tightness.
- (n) Pumping is resumed at a steady pace, the aim being to maintain the same rate of increase in both manometers. The elongations are read at both ends at pressures of 100, 200, 300 kg per sq. cm. and at the design pressure. These are recorded on

the spot and the total elongation worked out as illustrated below :

Beam No. 9C	Pressure	354 kg per sq. cm.
Cable No. 3	Elongation	255 mm

Pressure kg per sq. cm.	Elongation as read at			Correction Elongation at 2000 3	Corrected elongation
	Upper Block	Lower Block	Total		
50	...	...	...	36	36
100	20	16	36	...	72
200	70	39	109	...	145
300	109	72	181	...	217
354	136	86	222	...	258

In the case illustrated above, the final elongation reached is 258 mm at design pressure against the calculated 255 mm. This is satisfactory. Sometimes the requisite elongation may not be reached at the design pressure owing to excessive friction in the cable sheath. In such a case the stressing may be continued till the specified elongation is attained, subject to the limitation that a pressure of 380 kg per sq. cm. is not exceeded in the process.

Sometimes the specified elongation may be reached at a pressure less than the designed one. This indicates little friction between cable and sheath. In order to avoid over-stressing of the beams, such a condition must be detected from the trend of elongation and the stressing of the cable stopped when the specified elongation is attained.

- (o) Closing valve 1, Fig. 25, page 218 connecting the blocking chamber to the pump through inlet, *F*, and opening valve 2 by which the manometer pressure falls to zero, pumping is done to actuate the blocking ram which forces the inner cone into the anchorage. The blocking pressure is usually kept equal to the final tensioning pressure.
- (p) Valve 1, and then 3, are opened very gradually till the pressure falls to zero. The cranes are operat-

ed to hold the jacks and prevent them from slouching. The hose connections are detached from the jacks.

When the pressure is released completely, the elongation is measured. This measurement will indicate a small slip of the cable as a whole due to the inward movement of the inner cone during locking of the anchorage.

- (q) When the cylinder and ram have returned to normal position, the wedge grips are removed one by one by means of a pin hammer and sledge hammer, the pin hammer being held against the ledge of the wedge, and the sledge hammer delivering a sharp blow on the pin hammer. The operation is a bit delicate, but in skilled hands a single clean sharp blow will release the wedge.

The elongation is measured again. A very small slip of a millimeter or two of the entire cable may be noticed now.

- (r) The jacks are pushed back out of the wires and hoisted to the next cable. The cable wires are tied together at the ends with binding wire and the ring and sleeve removed for the next cable.

16.6. Measurement of Deflections

Deflectometers are mounted at the upper and lower end blocks and at the diaphragms. The deflectometers used are Baty Dial Gauges which read to an accuracy of 1/2000 in. The change in the flexure of the beam is recorded cable by cable by taking the dial readings before and after tensioning each cable. The observed deflections have been found to tally fairly well with the calculated values.

16.7. Some Essential Precautions

16.7.1. The only purpose of the steel sheath is to prevent contact between cable and concrete. Minor breaks in the sheath easily permit the ingress of matrix which will block the cable. So, after placing and lining up the cables a careful examination must be made for local damages. Damages, if any, must be sealed with adhesive tape.

16.7.2. Particular attention must be paid to the inner vertical faces of end blocks. The sheaths must be made long

enough to penetrate a few inches into the end blocks and gaps round the sheath must be caulked with stiff mortar or mastic. Places where cables enter anchorage cones in the top or bottom flange are also vulnerable. Cones with the projecting collar are more suitable, since the collar facilitates a tight connection with the sheath.

16.7.3. In spite of all these precautions some mortar may still find its way into the cables and anchorage cones. The mortar that collects in the anchorage cones is washed out before it has taken initial set by means of a spout of water under pressure. The mortar, if any, that may enter the cables is prevented from developing bond with the wires by pushing the cables forward and backward a few times inside the sheath by means of hammer strokes applied about 2 to 3 hours after the concreting. These measures have been found to be quite effective in freeing the cable of any bond.

16.7.4. The inner face of the outer cone, if covered with rust, dirt, etc., must also be thoroughly cleaned just before prestressing.

16.7.5. The anchorage zone of the cables, a length of about 2 ft on either side of the anchorage cone, must be dry and free from grease, oil, accretions of mortar, rust and all such foreign matter. In order to ensure this condition the following procedure is followed. The cable is hammered out from the upper end block side till about 2 ft more of the cable is exposed on the lower end block side. The entire exposed length, about 4 ft, is then thoroughly cleaned, but no attempt is made to polish the wires. Then the cable is hammered from the lower end block side till about 4 ft is exposed on the upper end block side. After this portion is also cleaned likewise the cable is hammered to normal position with about 2 ft projecting at each end. This cleaning should be done only just before prestressing.

16.8. Obstructions in Cable

If in spite of all these precautions faithfully carried out, cables get blocked, the following are some of the methods by which the obstructions may be removed.

(a) When the elongation is in jerks, the cause may be traced to some minor obstruction causing additional friction at some point between the anchorage cone and the point of minimum stress in the cable. While tensioning the cable, there is a small elongation accompanied by a sharp rise of pressure in the manometer until the extra friction is overcome. Then the cable

suddenly elongates with a jerk and the manometer shows a simultaneous fall of pressure until it stabilizes a little above the point from which it had started to rise sharply. Then the whole cycle is repeated again. Thus, when the obstruction is of this type, it may be possible to get the necessary elongation with perhaps a small increase of pressure over the design value. Very often, in the later stages of tensioning, the effect of the obstruction may completely disappear.

(b) When there is a severe obstruction near the anchorage cone, the manometer shoots up to values even as high as 200 kg per sq. cm. with a few strokes of the pump and there is no elongation in the cable. Keeping careful watch on the manometer so as not to raise the pressure above 380 kg, the pump may be operated at a slow and steady pace till the cable is released. When that happens, the cable will elongate with a jerk and the pressure will drop to perhaps a few kg per sq. cm. Thereafter, the tensioning may be resumed in the usual manner. If it is not possible to free the cable even after reaching a pressure of about 380 kg per sq. cm., pumping should be stopped and the specified elongation obtained by operating only the jack at the other end of the cable.

(c) If the obstruction happens to be very close to the anchorage, once the jack becomes self-carrying, the slightest flicker of the pump handle will cause the manometer to shoot up. In this case, also, the procedure mentioned in the preceding paragraph may be adopted but infinite care should be exercised to ensure that the pressure does not exceed about 380 kg per sq. cm. When the obstruction is severe and too close to the anchorage, it is safer to obtain all the elongation possible at the other end of the cable.

16.9. Leakage During Prestressing

16.9.1. If leakage develops in the tensioning cylinder or its connection while the cable is being tensioned, the pressure may be released and the jack taken down for repairs. Owing to the self blocking nature of the cone anchorage, the cable will still be in tension after some initial loss of elongation due to slip. The tensioning can be resumed with the spare jack which should be kept in good working trim always.

16.9.2. As a rule, the blocking pressure should be equal to the final tensioning pressure. On one occasion a cable was tensioned to a manometer pressure of 370 kg per sq. cm., but during blocking it was found that the blocking ram had suddenly developed a leak when the pressure was only 240 kg per sq. cm.

It was not possible to raise the pressure above 240 kg per sq. cm. in spite of fast pumping. So, while pumping thus, valve 1, Fig. 25, page 218, was quickly opened thereby effecting connection between the tensioning chamber in which the pressure was 370 kg per sq. cm. and the blocking chamber in which the pressure was only 240 kg per sq. cm. This caused the pressure in the blocking cylinder to rise momentarily until hydraulic equilibrium was established in the system, and effect a complete locking of the anchorage.

16.10. Troubles Arising from Defective or Worn Out Wedge Grips.

16.10.1. The wedges which secure the wires to the jack are subjected to severe wear. In a wedge which is in good condition, the groove on either side will have a straight and sharp V-shape. As wear sets in, the V-shape will change to a crescent shape and individual wires may show a tendency to slip during prestressing. Wedges should be replaced as soon as signs of wear are visible. Worn out wedges can be set right by sharpening the grooves to restore the V-shape. This involves removal of some material every time, so after a few sharpenings the size of the wedge will have been reduced beyond use. An adequate supply of serviceable wedges is very essential for the efficient performance of prestressing operations.

16.10.2. Relative slipping of individual wires during prestressing is only the earliest and the least harmful of the troubles arising from worn out wedges. Relative slipping may also occur when a wedge is not tightly hammered in even though it may be in first class condition. In either case the net result is a redistribution of loads between wires leading to a smaller load in the wire which has slipped and an increase of load in the remaining wires, the total cable force remaining the same. The tensioning may be continued to the design pressure, but because of the higher intensity of stress there should be more elongation than if there had been no slipping. Overstressing of steel is the danger to be guarded against. That this danger is quite real will be clear from the following example :

Suppose the design pressure is 380 kg per sq. cm. This corresponds to a tensile stress of $\frac{380 \times 160.6}{461} = 133$ kg per sq. cm.

If one of the twelve wires slip and loose all its elongation towards the end of the tensioning process, the steel stress in the cable will be $133 \times \frac{12}{11} = 145$ kg per sq. cm. or 92 ton per sq. in., which is too near the ultimate strength of 95—110 tons per sq. in.

16.10.3. Wires may slip due to worn out wedges at any stage during tensioning, not necessarily at maximum pressure. It is also possible that they may not loose all the elongation. If the slip takes place at some intermediate stage, the slipped wires also share the cable load in some measure depending on the amount of slip and the time of its occurrence.

16.10.4. Immediately a slip takes place, there is bound to be a fall of pressure. This is because the sudden increase of stress in the wires, which are intact, causes an increase of elongation pushing the tensioning cylinder back instantaneously and thereby causing a fall of pressure in the cylinder until equilibrium is established between jack and cable.

16.10.5. In attempting to carry out prestressing with wedges which are worn out there is always the risk of the wedges themselves flying off. Then the wires suddenly released may get bent near the other jack due to the rebound of the cable. When the pressure in the jack is released for taking them down and preparing the cable for retensioning, the anchorages may lock. Owing to the violence and suddenness of the process the inner cones are liable to be damaged. As such, they should be removed and the cable completely released before redoing the tensioning after straightening the wires that are bent, if any.

16.10.6. Removing the inner cone from a locked anchorage is a delicate operation. It may be done with the aid of the prestressing jack in the following manner. The face of the inner cone is chipped off uniformly for about $\frac{1}{2}$ in. depth to expose the flared end of the grouting tube. A piece of binding wire is tied round the grouting tube and on mounting the jack the ends of the wire are pulled out through the grooves in the foot of the jack and held firmly. The jack is operated till the inner cone becomes free when it is pulled back sufficiently by means of the wire and secured in that position so that on releasing the jack pressure the retreating cable will not cause the anchorage to lock again. On taking down the jack the inner cone is easily pulled out.

16.10.7. After the anchorage is locked tight by operating the blocking ram at the end of the tensioning process, individual wires should not slip on dewedging, but sometimes this may happen owing to interference in the anchorage or due to the use of worn out wedge grips. Slipping of wires after dewedging leads to permanent loss of tension depending on the amount of slip, and thus leads to consequences very different from those due to slipping of wires during prestressing. If only one wire has slipped, it may be possible to make good the loss of tension by slightly overstressing other cables. But when the circumstances

are not favourable for such manipulations, it is necessary to re-tension the cable after removing the outer cone.

16.11. Possibility of Wires Breaking

As a rule, during prestressing the jack pressure is not raised beyond 370 kg per sq. cm. except in exceptional circumstances when it may be raised to 380 kg per sq. cm. This corresponds to a tensile stress of 133 kg per sq. mm. or 84.5 ton per sq. in. and is well within the ultimate strength of the steel. Breaking of a wire may be expected to take place when the maximum pressure has been reached and before locking the anchorage, since locking of the anchorage is accompanied by a loss of tension. In the whole prestressing operation only in three instances did the wires break, but strangely enough in all the three cases the pressure was only about 330 kg per sq. cm. corresponding to a tensile stress of only 73 tons per sq. in. Since the ultimate strength of the steel could not have been as low as this it has not been possible to explain this. In all the cases, the wires broke close to the wedge grips, which is but natural since the stress is maximum near wedges. The catapulting action due to the breaking of the wire was so violent that it blew off three or four wedges of the jack at the other end and the jack itself received a violent kick. Some wires were also badly bent. The inner cones had to be removed and the cable tensioned again with only eleven wires. The maximum pressure was limited to 11/12 of the design pressure so as not to exceed the design stress in the cable. The small resulting deficiency in the prestressing force was made good by slight overstraining of some cables tensioned later.

16.12. Check of Prestress at Critical Sections and Adjustments to Ensure Proper Values

Ten cables, No. 1 to 10, pass through critical sections in the beam and twelve cables, No. 3 to 12, 15 and 16, in the cantilever as shown in Fig. 9. This applies for spans 2 to 12, the cabling being different for the other spans, but the method of check given below applies to all cases.

The theoretical strains and elongations are noted in columns 5 and 4 of Table 6. The actual elongations are noted in column 3 as each cable is tensioned and the actual strain per milli is worked out by rule of proportion and entered in column 6. The difference between actual and theoretical strains is entered in the plus or minus columns, 7 to 10.

For example, in cable 7, the actual strain is more than the theoretical by 0.04; as this cable passes through both beam and cantilever critical sections, 0.04 is entered in both the plus

TABLE 6
Theoretical and Actual Strains and Elongations
Type II Beams

Order of tensioning	Cable No.	Elongation		Strain per milli		Difference of strain actual to design			
		Actual	Design	Design	Actual	Beam		Cantilever	
						Plus (+)	Minus (-)	Plus (+)	Minus (-)
1	2	3	4	5	6	7	8	9	10
1	7	248	246	5.00	5.04	.04	...	.04	...
2	5	253	253	5.15	5.15	...	...	...	...
3	3	256	255	5.10	5.12	.02	...	.02	...
4	6	250	253	5.15	5.08	...	.07	...	.07
5	4	262	255	5.10	5.24	.14	...	.14	...
6	8	247	245	4.95	4.99	.04	...	.04	...
7	9	247	244	4.95	4.61	...	.34	...	.34
8	10	246	244	4.95	4.58	...	.37	...	.37
9	1	145	145	5.45	5.45	...	...	...	...
10	2	158	157	5.45	5.48	.03	...	...	...
11	11	71	72	5.00	5.07	...	...	.07	...
12	12	72	72	5.00	5.00	...	...	...	...
13	15	142	134	5.05	5.35	...	...	.35	...
14	16	144	137	5.10	5.35	...	...	.25	...
Total for cables 1-10 (Beam)				51.25		.27	.78		
Total for cables 3 to 12, 15 & 16 (cantilever)				61.50				.91	.78

$$\text{Difference for beam} = -0.51 = \frac{-0.51}{51.25} \times 100 = -1 \text{ per cent}$$

$$\text{Difference for cantilever} = +0.13 = \frac{0.13}{70.40} \times 100 = +0.2 \text{ per cent}$$

columns 7 and 9. If the cable passes through only one critical section, then the entry is made only in one column—cable 2 for example.

As tensioning proceeds cable by cable, the plus and minus columns 7 and 8 or 9 and 10 must cancel out as far as possible. If either sign has a tendency to dominate, the tensioning of other cables must be adjusted to counteract this tendency. This adjustment may not be possible to perfection, but a great deal of accuracy can be achieved by keeping careful watch. It is sufficient for practical purposes if the final cumulative difference does not exceed 5 per cent of the total theoretical strain. In the case under illustration, the difference is only -1 per cent for the beam section and +0.2 per cent for the cantilever, and as such, quite satisfactory.

16.13. Friction Losses in Cables

16.13.1. In the evaluation of elongation and manometer pressure for prestressing it has been necessary to calculate the tension at cable ends and the minimum tension after friction loss along the length of the cable. The elongation has been calculated for the average tension T_m obtained by the formulae for parabolic cables, viz.,

$$T_m = \frac{2T_{min.} + T_{max.}}{3}$$

The theoretical friction loss is $T_{max.} - T_{min.}$

The actual mean tension may be derived by multiplying the theoretical value by the ratio of actual to theoretical elongation thus—

$$T_m \text{ actual} = T_m \times \frac{\text{elongation actual}}{\text{elongation theoretical}}$$

$$T_{max.} \text{ actual} = \text{Applied manometer pressure} \times \frac{160.6}{461}$$

These actual values of T_m and $T_{max.}$ may be made use of to calculate $T_{min.}$ actual by means of the formulae for average stress: Thus—

$$T_{min.} \text{ actual} = \frac{T_m \text{ actual} \times 3 - T_{max.} \text{ actual}}{2}$$

Then the actual friction = $T_{max.} \text{ actual} - T_{min.} \text{ actual}$.

By way of illustration, take cable No. 7 in beam 10-a. We have the following particulars:

Design Values

Manometer pressure	346 kg per sq. cm.
Elongation	246 mm
T_m	103 kg per sq. mm.
$T_{max.}$	120 kg per sq. mm.
$T_{min.}$	94.5 kg per sq. mm.
Friction loss	$120 - 94.5 = 25.5$ kg per sq. mm.

Actual Values

Manometer pressure	355 kg per sq. cm.
Elongation	247 mm
T_m actual	$103 \times \frac{247}{246} = 103$ kg per sq. mm.
$T_{max.}$ actual	$\frac{355 \times 160.6}{461} = 124$ kg per sq. mm.
$T_{min.}$ actual	$\frac{3 \times 103 - 124}{2} = 92.5$ kg per sq. mm.
Friction loss	$124 - 92.5 = 31.5$ kg per sq. mm.
Excess of actual friction over design value	$31.5 - 25.5 = 6$ kg per sq. mm.
Per cent excess	$\frac{6}{25.5} \times 100 = 24$

16.13.2. In order to study the variation between actual friction and friction as provided for in the design, the case of cable No. 7 in 31 beams has been examined by determining the theoretical and actual friction as explained and illustrated above. The study reveals that the actual friction varies from the theoretical value by as much as +53 per cent and -41 per cent in extreme cases with the average for the 31 beams at +6 per cent. The relevant figures are:

Loss in Friction

Theoretical	25.5 kg per sq. mm.
Actual maximum	39.0 „
Actual minimum	15.0 „
Average of 31 beams	27.0 „

It will be noticed that in spite of the wide scatter the average for a large number of results reveals a fairly close approximation to the theoretical value allowed for, thus justifying the empirical formulae adopted in the calculations.

16.13.3. There were two beams in which the cables had to be stressed from one end only owing to one of the jacks going out of order (and at that time there was no spare jack). In these beams, for the cable No. 7 under study, the friction losses shot up to 43.5 kg per sq. mm. and 45 kg per sq. mm., clearly proving the disadvantages of stressing long cables from one end only.

16.14. Loss of Tension due to Locking of Anchorage

16.14.1. For cables with cone anchorage at both ends, the loss of elongation due to slip on locking of the anchorage by ramming in the inner cone has been as low as 10 mm and as much as 21 mm, but in the vast majority of cases the total slip at both ends has tended to remain within 16 mm. Cable length alone does not appear to influence the value, but cables with relatively sharp curvature, viz., No. 11 and 12, even though short, have a tendency to slip less than the longer cables with less curvature.

16.14.2. For cables with cone anchorage only on one end, viz., No. 15 and 16 in typical beams, the total slip has tended to remain well within 8 mm, although in a few exceptional cases this value has been exceeded.

16.14.3. In calculating the prestressing, no allowance has been made for loss of tension due to locking of the anchorage, because it has been stated to be significant only in the case of short beams. As an illustration, taking cable No. 7 in beam 6 d, after blocking, there was a loss of 8 mm, at the lower end block and 9 mm at the upper end block. It is assumed that the cable is absolutely free to adjust itself, i.e., if it is free from friction, then the loss of tension would be—

$$\frac{17}{48.5 \times 1000} \times 20,550 = 7.2 \text{ kg per sq. mm.}$$

48.5 metres being the length of the cable, and 20,550 kg per sq. mm. the value of E_s . But actually there is considerable friction in the cable, as much as 21 kg per sq. mm. being the tension loss due to friction. This friction which was obstructing the tensioning process is advantageous in resisting slipping of the cable during locking of the anchorage and prevents the loss due to the slip being felt in the critical section.

16.14.4. A very small slip, 0–4 mm, may be noticed after the dewedging. On any one side ordinarily it will not exceed 2 mm. This is largely made up of the movement of the reference mark on the wire due to the relaxation of tension in the cable between the anchorage face and the wedge grips of the jack. This would be noticeable only if, at the time of measuring the elongation after blocking, there was some pressure in the jack and therefore some residual tension in the short length of cable mentioned above. If the elongation was measured after blocking and after the jack has completely returned to normal, no slip should be noticeable after dewedging.

17. LAUNCHING THE BEAMS

17.1. General

The weights of the three types of prestressed beams are given below:

	Tons
Type I	120
Type II	100
Type III	80

Their shape, height, and narrowness of bottom flange are not conducive to stability.

17.2. Launching of Beams in Spans 1 to 3

17.2.1. A launching girder which moves ahead of the spans for which girders have already been launched and rests over the span for which beams are to be placed in position, is used. It serves as a bridge to transport the beams.

Even though orders were placed for the supply of the launching girders early, there was no prospect of receiving them for use in time. So, an alternative plan for launching the girders of spans 1 to 3, by making use of some of the transport cars, shifting beam, tail piece, rollers, etc., which were available in time, was evolved.

17.2.2. In the first 2 spans, a broad gauge double track of 41½ lb rails was laid over a sand embankment for the transport car carrying the beam. In order to limit the pressure on the embankment within safe limits, the tracks in the first and second spans were laid on a double layer of broad gauge wooden sleepers. The first layer of sleepers was closely packed longitudinally. The second layer was laid transversely at about 2 ft

centres to function as ties and distribute the load over the first layer. In the third span, the arrangement was slightly different.

17.2.3. Through a gap left temporarily in the curtain wall of the north abutment, the main trolley track from the casting yard was extended for about 70 ft over what may be termed as the "transshipping yard" where the transport cars took over the beams from the trolleys. The position of transport car I, with reference to the bearing, *G*, of the beam was such that when the car came to a stop against the pier, the bearing would be in the correct position with reference to the centre line of the pier.

17.2.4. For launching the girders of span No. 2, transport car I was dismantled and reassembled on the track in the bed of span No. 2. Transport car II was assembled on the broad gauge tracks laid over the beams already in position in span No. 1. On approaching the transshipping yard, the beam was picked up at the haunch by transport car II and moved along with the rear of the beam supported on a trolley. After clearing pier No. I, the beam was set down over transport car I waiting on the far side of pier No. 1. Transport car II was then rolled back to pick up the rear end of the beam from the trolley. The cars then moved forward with the beam, Photo 27. The limiting position for this arrangement was reached with transport car II at the end of the cantilever in span 2. For the few feet still to travel the beam was moved with its rear end supported on a trolley working over a specially laid track.

17.2.5. After launching the girders of the second span, transport car I was again dismantled and reassembled on the bridge. The tracks on the bridge were extended over span No. 2. In span No. 3, a trolley track was laid on a well compacted embankment, the top 3 ft of which were built up solidly with sand cement blocks laid over a layer of closely packed sleepers. The beams were carried to the end of the cantilever by means of the transport cars, Photo 28, and then transferred to trolleys working on the track in the third span which carried the beams to their position.

17.2.6. For moving the beams forward from the casting yard, a 10-ton winch was set up at the end of the transshipping yard. For moving the beams from there to the spans, a 10-ton winch was set up on a pier one span ahead of the span for which girders are to be launched. For the lifting and lowering of the beams hydraulic jacks were used.

17.2.7. For moving a beam transversely to its correct position it was set down on a "shifting box" over two roller grids

on a "sliding plate" kept on the pier, Photo 29. The rear end of the beam was set down on two other roller grids over a "shifting beam" placed across the end of the cantilever in the span, the latter grids being reached through a tail piece attached to

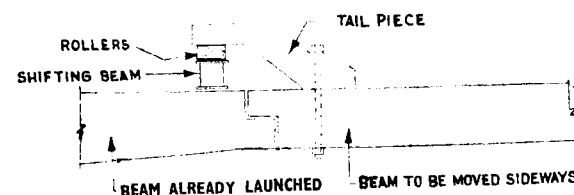


Fig. 27. Tail Piece for shifting the beam sideways

the rear end of the beam, Photo 30 and Fig. 27. The beam was then pulled sideways by means of two differential pulleys, each of 1½-ton capacity, one anchored to the pier and the other to the shifting beam itself.

17.2.8. The beams were accurately established in their final position on precast bed blocks over piers and rubber plates at the hinge by a theodolite set up on the pier two spans ahead so that the transverse cable ducts in the top flange matched with similar ducts in the adjoining beam.

17.2.9. The criteria for the number of rubber plates to be inserted at the hinge were that the soffits of the beams meeting at the articulation and the transverse cable ducts should match. There should be a minimum of five plates for expansion and contraction, but a larger number of plates up to seven were found necessary in some cases; by and large, five plates, on an average, were found to give optimum satisfaction from all considerations.

17.2.10. There were instances when the horizontal clearance between the two beams meeting at an articulation, was found to be either too much or too little. In such cases, the beam had to be moved forward or backward to rectify the error before establishing the beam in its final position. For this correction the equipment for side shifting was used. Photo 31 shows an articulation which carries a rubber plate bearing at the hinge.

17.3. Launching with the Aid of the Launching Girder

The launching consists of the following stages:

- (a) Moving forward the launching girder over the span last completed, on to the next span and setting it down in working position.

- (b) Transport of beams to the transshipping yard at the north abutment by trollies.
- (c) Transport of beams over completed spans by transport cars.
- (d) Transport of beams along the launching girder by travelling crabs.
- (e) Side shifting of beam and lowering on bearings.

17.4. Launching Girder and its Erection

The launching girder is a V-shaped truss consisting of 6 bays of 16 ft each and 8 bays of 17 ft each, Fig. 28, with detachable connecting boxes at 0 and 6 to facilitate the attachment of

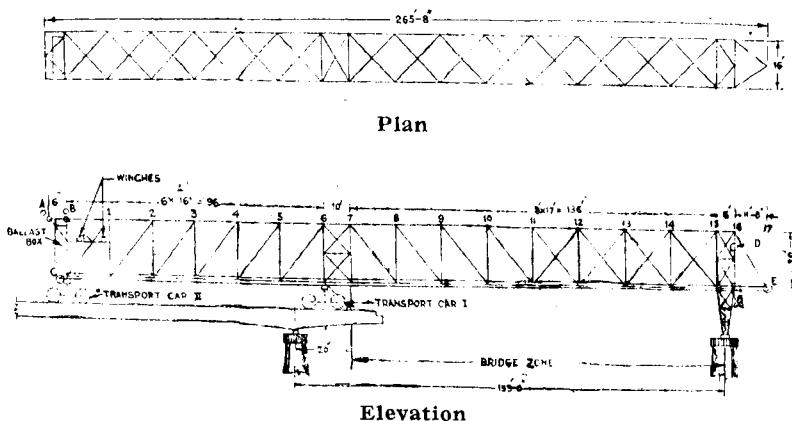


Fig. 28.

the girder to transport cars. At 7 and 15-16 are standards upon which the girder rests in working position. The bottom boom of the girder is designed and shaped to function as a mono-rail along which the travelling crabs move forward carrying a beam. The top booms are connected by ties to resist stresses induced by wind loads. At the rear of the girder is a large ballast tank which is filled with water for stability when the girder is moved forward with the transport cars attached at 0 and 6 and with the rest of the girder - the bridge zone - cantilevering from 6. Two 2-ton winches, one for moving the launching girder forward after completing a span, and the other for hauling the beams along the mono-rail bottom boom, are mounted on a platform in the first bay; pulleys are provided at appropriate places on the girder to guide the hauling lines from the winches. The top booms are 16 ft apart and the height of

the girder from bottom boom to top boom is 20 ft. In working position, the bottom boom is about 17 ft 9 in. from the top of piers. The total length of the girder from end to end is 265 ft 8 in. and it weighs 58 tons.

17.5. Erection of the Launching Girder

After the structural parts were identified, sorted out by the match-marks and arranged in the sequence in which they would be required, the bottom chord pieces were joined up by derrick one after the other. The first piece was connected to the transport car II nearest the north abutment and held in position by temporary supports. Other pieces were added on and held in position by temporary supports, and at point 6 the transport car I was connected. The "verticals" (sides of the V) were then hoisted by derrick, connected to the bottom gusset and held in position by temporary supports. As soon as a sufficient number of "verticals" were in position to cover the first shop length of the top chords, the top chords were hoisted and connected with the "verticals." The diagonal members in the top and web members were next put in. Photo 32 shows the erection in an advanced stage. All field connections were of bolts. The bolts were tightened to refusal by means of long-handled spanners.

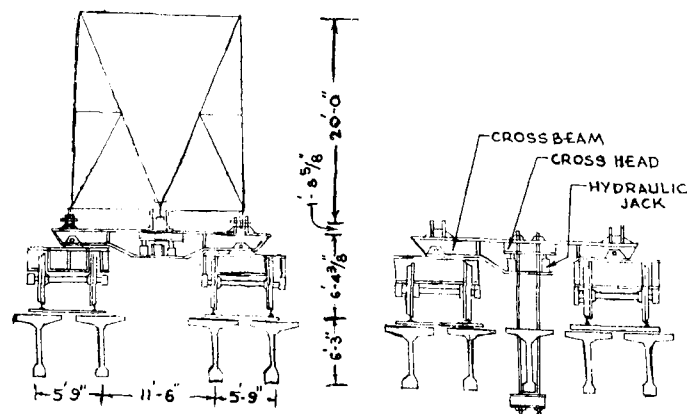
17.6. Transport Cars

17.6.1. The two transport cars constitute the main rolling stock of the launching system. A transport car is made up of two trollies, each constructed of a pair of broad gauge axles and wheels; the trollies are connected together by a cross beam with a pin connection at each end, the arrangement being designed to secure an equal distribution of load in all the wheels. The cross beam consists of two built up I-sections of variable depth connected together in the webs by means of stiffener plates. In the central compartment, on shelves provided for the purpose, are two hydraulic jacks which can be operated to raise or lower a cross head placed over the jacks. Through holes at the ends of the cross head, rods pass down the sides of a beam and are secured by a clip as shown in Photo 28. When the pressure in the jacks, which are connected to a common pump, is raised sufficiently, the beam is lifted up by the suspension device. Spherically seated nuts and washers securing the rods at the top permit the suspension device to take up slight side or longitudinal sway without harm to itself.

17.6.2. The jacks in the forward car I are of 50 tons capacity each and those in the rear car II are of 25 tons each. The trollies of this car are fitted with an undercarriage to accom-

modate some concrete ballast which is necessary when moving the launching girder.

17.6.3. The transport cars travel on a pair of broad gauge tracks, laid over the completed spans, Fig. 29. It may be seen



Transport cars moving the launching girder

Transport cars moving a beam

Fig. 29 Transport cars

that the beams have been paired with a spacing of 5 ft 9 in. between an inner and an outer beam so that when rails are laid along the centre line of the beam the correct gauge will be secured.

17.7. Travelling Crabs

17.7.1. These are the devices used for moving a beam along the mono-rail (bottom chord) of the launching girder after the beam is brought to the limit of completed spans. There are three crabs in all: for convenience these may be designated as crab I, II and III. Crabs I and II comprise one unit for supporting the beam near the bearing, *G*, and crab III for supporting it near the upper end block at the rear end of the beam. Each crab consists of sixteen 10-in. wheels mounted on four U-frames, coupled together by means of pin joints. Suspended from the frames connecting the U-frames are two A-shaped frames individually pin-jointed to a chain carrying two 25-ton hydraulic jacks. The hydraulic jacks support a cross head and suspension device similar in design to that in the transport cars, Photo 33. The four jacks in crabs I and II are connected to one common pump, and the two jacks in crab III are connected to another pump. Thus each set of jacks may be raised or lowered in unison maintaining equal pressure.

17.7.2. The elements of the crab being pin-jointed, any longitudinal sway of the load during movement is accommodated without disturbing the uniform distribution of wheel loads. In order to ensure this as well as to avoid any bending action on the suspended rods due to side or longitudinal sway, the nuts and washers supporting them from the cross head are provided with a spherical seating permitting a slight pendulum action of the rods. During the movement of the crabs, the jacks are kept under pressure so that the beam virtually "floats" and thereby any possibility of an unequal distribution of loads between the trains of wheels on the left and right flanges of the mono-rail is completely eliminated.

17.8. Stability of Launching Girders

17.8.1. While moving, the launching girder is supported on transport cars I and II at points 6 and 0 and the portion of the girder from 6 to 15-16, functions as a cantilever. The approxi-

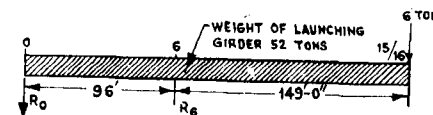


Fig. 30.

mate distribution of the weight of the girder and the reaction at 0 and 6 are as shown in Fig. 30. Taking moments about point 0:

$$R_6 \times 96 = 6 \times 245 + 52 \times \frac{245}{2}$$

from which

$$R_6 = 82 \text{ tons} \uparrow$$

$$R_0 = 82 - 58 = 24 \text{ tons} \downarrow$$

Stability for moving the launching girder cannot be achieved unless a minimum downward reaction of 24 tons is ensured at point 0. A total downward force of about 30 tons at point 0 is provided by the weight of the ballast tank filled with water, and the weight of the transport car II with concrete ballast; the car is, of course, properly secured to the girder in order to develop the anchorage effect.

17.8.2. In order to bring the cantilever 6 to 15-16 into action, the girder must be jacked up at point 6 until 15-16 is clear of the beams previously launched. In this matter the deflection of the girder plays a significant part.

The theoretical deflection of 15-16 relative to point 0 is 12.5 in. In bay 8-9, the top chord is shortened by 1.19 in. to counter

this deflection partially. The result of this chord-shortening is to lift up 15-16 by 6.1 in., but this is reduced by 3.4 in. due to some inadvertent error in the length of the bottom chord making it short by 0.5 in. in the bay 6-7. So, the net downward deflection is 9.8 in. theoretically. This indicates that point 6 should be jacked up by about 3.8 in. above point 0 for 15-16 to clear the beams.

17.8.3. When the launching girder was sought to be put into operation for the first time, the anticipated reaction of 82 tons at point 6 did not develop and 15-16 did not lift clear of the beam even though point 6 was jacked up by nearly 5 in. above point 0. Many anxious days were spent in checking the design of the girder and calculating and re-calculating the deflections. Still theory and practice did not tally.

With the points 0, 6 and 15-16 at practically the same level, the reaction at 15-16 was measured by a hydraulic jack and it was found to be 34 tons. For every inch of height through which the point 15-16 was lowered, the reaction decreased by 1.74 tons. Hence, if the point 15-16 was free to deflect, it might be expected to be free when it had come down by $34/1.74$ in., that is about 20 in. This implied that point 6 should be jacked up about 8 in. above point 0 if point 15-16 were to be lifted clear. Subsequent results proved this prediction correct. When point 6 was jacked up 8 in. above the level of point 0, R_6 developed a value of about 83 tons and 15-16 lifted clear of the beams.

17.8.4. The excessive deflection of the structure could be attributed to imperfections in the field joints due to their being bolted. Bolted connections could not, perhaps, develop the friction that a rivetted or welded joint could. Besides, each shop length of bottom chord was given a certain amount of camber during manufacture in order to offset any permanent sag it may develop under load. In the case of the connection at point 6, the fixed end of the cantilever, this camber appeared to give the entire cantilever a slight downward tilt.

17.9. Maximum Stresses in Beams during Launching

17.9.1. The following cases have been examined :

Case 1. When the launching girder moves from span n , the last completed span, to working position over the next, the transport car I at point 6 exerts a reaction of 82 tons or 186 kips on four beams. The net downward reaction exerted by the transport car II at point 0 is about 6 tons or 13.5 kips. The maximum negative moment is developed in span $(n-1)$ when transport car I is at the hinge in the span, n ; in this

position the loading of a beam in span $(n-1)$ is shown in Fig. 31(a).

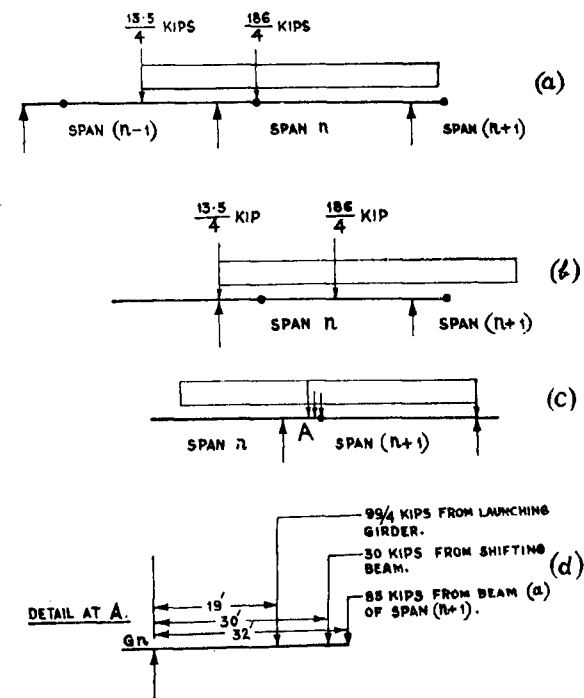


Fig. 31.

Case 2. The maximum positive moment is developed in span n as the transport car I passes the hinge and travels over the span n , Fig. 31(b).

Case 3. After traversing the span n , the launching girder comes to rest in the next working position with point 7, about 19 ft from G_n . Then it exerts a reaction of about 99 kips on the beams. The beams are launched in the sequence a, b, d, c . After beam 'a' of span $(n+1)$ is launched, chances of overloading the cantilever occur during the side shifting of b, d and c of span $(n+1)$. By observing the precautions enumerated below, the chances of overloading can be minimised. When side shifting b of span $(n+1)$, the shifting beam should not bear on cantilever a , but should be carried only on cantilevers b and c , or b and d . When side shifting d of span $(n+1)$, the shifting beam must of necessity be supported on b and d or even on b, c and d . In either case, b has to bear a large reaction during the side shifting, besides carrying the reaction from the new beam b of span

$(n + 1)$ already launched. However, the effect of this is easily counteracted by loading span n with beam c of span $(n + 1)$ keeping transport car I with this beam near point S_2 , in which position it exerts the maximum positive moment. In side shifting beam c of span $(n + 1)$ the position is rather delicate; the excess load on the cantilevers is kept down to a third of the reaction from this beam by supporting the shifting beam on cantilevers of a and c . Fig. 31 (c) and (d) show the resultant loading on cantilever a , which is the worst condition for this case.

17.9.2. The bending moments resulting from the pattern of loading described above in the three cases are set forth in Table 7 and the resultant maximum fibre stresses in Table 8.

17.9.3. The stresses produced in cases 1 and 3 mentioned above by the movement of the launching girder are transitory by nature and last but an instant. They happen only once for every beam; all the same there should be adequate security against cracking of the beams. It may not be possible to eliminate tensile stresses altogether, but they should be kept down. Transitory tensile stresses of relatively low magnitude such as those in the top fibres at S_2 and S_3 (Case 1) can do no harm; further, the presence of two cables running along the top flange from G to beyond S_2 will enable the beam to hold out against any cracking that may occur in this region.

TABLE 7
Maximum Bending Moments* Due to Launching

	S_1	S_2	S_3	G
<i>Case 1</i>				
1. Due to own weight	+1,960	+2,353	+1,502	-656
2. Due to next span	-680	-1,370	-2,050	-2,730
3. Due to launching girder	-318	-637	-1,116	-1,488
Total of case (1): 1+2+3	+962	+346	-1,664	-4,874
<i>Case 2</i>				
4. Due to launching girder	+1,170	+1,480	+1,170	—
Total of case (2): 1+4	+3,130	+3,833	+2,672	-656
<i>Case 3</i>				
5. Due to loads, Fig. 31 (d)	-340	-690	-1,030	-1,370
Total of case (3): 1+2+5	+940	+293	-1,578	-4,756

* All moments are for one beam,

TABLE 8
Maximum Stresses Due to Launching

	S_1	S_2	S_3	G
Prestress				
Top	-442	-257	+557	+1,806
Bottom	+2,760	+2,897	+1,237	-588
Case 1				
Top	+426	+153	-738	-1,465
Bottom	-650	-234	+1,128	+2,090
Resultant Stresses				
Top	-16	-104	-181	+341
Bottom	+2,110	+2,663	+2,365	+1,502
Case 2				
Top	+1,385	+1,700	+1,186	-197
Bottom	-2,120	-2,590	-1,810	+281
Resultant Stresses				
Top	+943	+1,443	+1,743	+1,609
Bottom	+640	+307	-573	-307
Case 3				
Top	+416	+130	-700	-1,434
Bottom	-635	-198	+1,066	+2,040
Resultant Stresses				
Top	-26	-127	-143	+372
Bottom	+2,125	+2,699	+2,303	+1,452

The tensile stresses in the bottom fibres of S_3 and G in loading case 2 are pretty large. On account of this, supplementary mild steel reinforcement is provided in this region.

In case 3, tensile stresses are developed in the top fibres at S_2 and S_3 . These stresses will last for the duration of the side shifting which may be an hour or so. However, it is possible to alter this state of loading favourably by the simple device of placing the transport cars between S_2 and S_3 during the side

shifting of beams of span $(n+1)$. The positive moment generated by this load changes the tensile stress at S_2 to a small compressive stress and reduces the tensile stress at S_3 to 63 lb per sq. in. only.

17.9.4. During the launching operations, careful observations were made to detect the presence of cracking that might be expected in accordance with the results of the theoretical analysis. Not even a single hair crack has been noticed. Since the loading during launching is more exacting than the design load, it is no exaggeration to say that every beam in the bridge is automatically pre-tested.

17.10. Moving Launching Girder to a New working Position

17.10.1. At the commencement, the launching girder rests on standards at 7 and 15/16, the former blocked up about 13 ft behind the hinge of the last completed span and 15/16 above G over the pier, Fig. 32 (a). The water ballast tank is empty, the guy ropes at 15/16 are tight, and the travelling crabs are in the

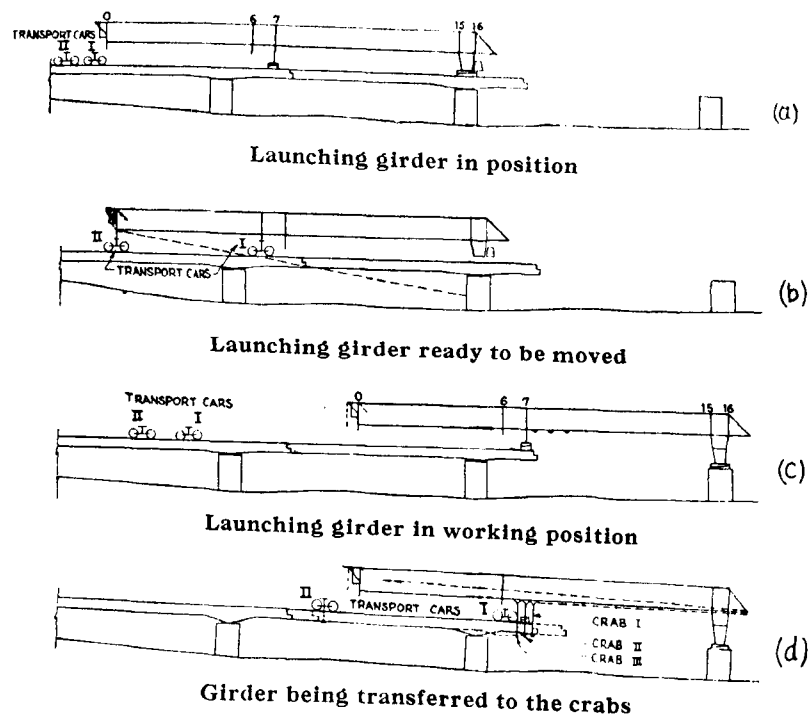


Fig. 32. Various stages in launching a beam

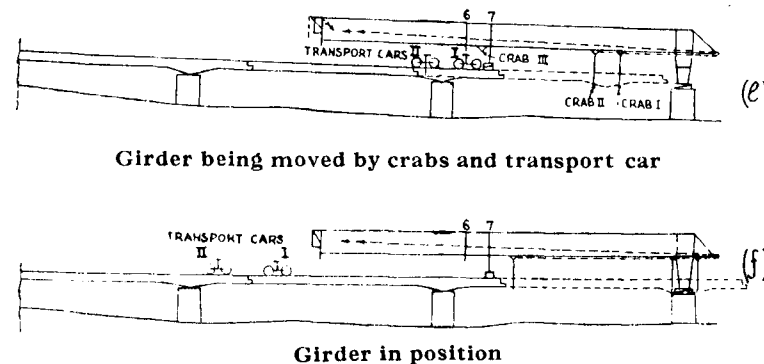


Fig. 32. Various stages in launching a beam

bridge zone between 7 and 15/16 with the rods of the suspension device dismantled.

17.10.2. The connecting boxes at 6 and 0 are now screwed on to the bottom chord and transport cars I and II are moved up to 6 and 0 and stopped by means of wooden clogs exactly under the respective connecting boxes. By operating the pump connected to the two jacks in transport car I, the launching girder is jacked up at 6 till standard 7 is free and the supporting girders under 7 are removed. The launching girder is then lowered by slowly releasing the pressure of the jack at 6 until the connecting box at 0 comes to rest on the transport car II, when the connection between this car and the launching girder is established.

17.10.3. The ballast tank is now filled with water, the guy ropes at 15/16 slackened and unhooked from their anchorages on the pier, the travelling crabs moved up as close as possible to point 6 and the launching girder jacked up at 6 until 15/16 becomes free and it is possible to remove the wooden blocks under 15/16. The zone 6 to 15/16 of the launching girder is now functioning as a cantilever. Liner blocks are next inserted to close the gap between the connecting box and cross beam of the car at 6 and the jack pressure released so that the launching girder rests on the transport car at 6. The end pins are also inserted to complete the connection.

17.10.4. The hauling line from the rear winch in the launching girder is connected to transport car I in the following manner. A $\frac{1}{2}$ -in. wire rope is used for the hauling line. It is passed round pulleys A, B, and C, Fig. 28, then doubled back round the single sheave pulley anchored to the next pier, and

hitched to a loop of wire rope hooked to the left and right trolleys of transport car I, Fig. 32(b). The launching girder is then moved along the tracks laid on the girders already in position by operating the winch, Photo 34. The forward movement is continued until the standard 15/16 is above the next pier; in this position the cars are again secured by clogs, the hinged part of 15/16 is lowered, Photo 35, and blocked up and the guy ropes hooked to the anchorages on the pier are tightened, Photo 36. The actual movement from one working position to the next takes a minimum of one hour.

17.10.5. At point 6 the girder is then jacked up to remove the liner blocks and pin connections and lowered slightly to reduce the reaction to about 50 tons; now the launching girder ceases to exert an upward reaction at 0. The connections between girder and the transport car II at 0 are released and the ballast tank emptied causing the launching girder to deflect upwards in the zone 6-0. The transport car at 0 may now be rolled back.

17.10.6. The two supporting girders are placed below the standard 7 over the rails of the track so as to span the distance of 5 ft 9 in. between the beams *a - b* and *c - d*. The launching girder is then lowered at point 6 through an inch or two till the standard 7 comes to rest on the two supporting girders, the left foot and the right foot of the standard establishing contact with the supporting girders simultaneously, thereby ensuring that the launching girder is level in cross section and transmitting loads equally to the left and to the right. If the supporting girders are at the same level and the left and the right supports of 15/16 also at the same level, the foregoing condition is easily satisfied. The transport car at 6 is now free to be rolled back. The hauling line is also detached and reeled up on the winch. The launching girder is now in working position, Fig. 32 (c).

17.11. Moving Beams to Transshipping Yard

The beam is moved by means of trolleys along the transverse track from the stock yard, until it is exactly above the longitudinal tracks. From pedestals at the junctions of the main and transverse tracks the beam is jacked up, the trollies on the transverse tracks released, and the trolleys on the main track moved to position under the beam at *G* and the rear end. The beam is then lowered on to the trolleys of the main track by releasing the jacks and moved forward to the transshipping yard by a winch. The transfer is effected one end at a time. During the transfer, the beam was checked for verticality by a plumb bob against reference lines on the diaphragms at *G* and at the upper end block.

If the beam is not vertical, a thin layer of fairly stiff cement mortar is spread over the support in a wedge shaped fashion to counteract the cant.

17.12 Moving Beam Over Completed Spans

17.12.1. When the beam arrives at the transshipping yard, the position of transport car I is adjusted so that it stands symmetrical with reference to the pair of holes *A*, Fig. 33, in the top flange of the beam. The cross head over the

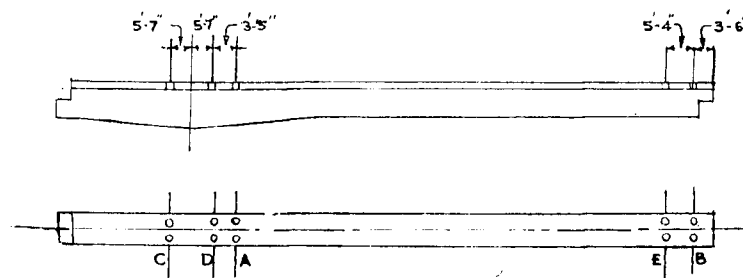


Fig. 33.

pair of jacks in this car is removed and the suspension rods are introduced through the holes using a light tripod and pulleys. The cross head and the clip at the bottom are then inserted and bolted on. It is to be ensured that the rods are truly vertical and the cross head and the clip truly horizontal. By operating the pump to which the two jacks in this car are connected the beam is lifted clear of the front trolley. The beam is moved forward along the tracks on the beams till the rear trolley reaches the transshipping yard. Transport car II is now shifted to be exactly above the pair of holes *B*, Fig. 33, and the rear end of the beam is picked up by the car.

17.12.2. To move the beam forward thus, a 2-ton winch is clamped to the top flange of the beam close to and behind the transport car I. The $\frac{1}{2}$ -in. wire rope from the winch passes over one of a pair of guide pulleys clamped to the end of the cantilever, doubles back by passing round a pulley anchored to a pier ahead, and is hitched to a loop of wire rope connected to the two trollies of the transport car I. To ensure the direct transmission of the tractive force from transport car I to car II and prevent any horizontal force on the suspension rods, the two cars are interconnected by a wire rope kept tight by means of a turn buckle. The movement can be accomplished at the rate of 150 ft per hour by eight men.

17.13. Moving Beam by Travelling Crabs along Launching Girder

17.13.1. As soon as the cross beam of transport car I passes point 6 of the launching girder, the cars are stopped and secured by wooden clogs, Fig. 32 (d). The forward crabs I and II are rolled back till they are exactly above the two pairs of holes *C* and *D*, Fig. 33. The suspension rods of the crabs are lowered through the holes by means of pulleys and connected by cross heads at the top and clips at the bottom. The four 25-ton jacks in the two crabs are connected to a single pump through a 4-way connection and the jacks are pumped till the beam reaction of about 65 tons on the transport car I is taken up entirely by the crabs, and the suspension rods of the car are hanging loose. These rods are then unscrewed and lifted clear of the beam disengaging the car. The winch clamped to the beam just behind the car is also released and hoisted on to the car to be returned to the abutment for the next beam. The beam is now moved forward along the launching girder with the rear end still slung from the transport car II, Fig 32 (c) and Photo 37.

17.13.2. To move the beam along the launching girder, the 2-ton winch in the forward position on the winch platform is

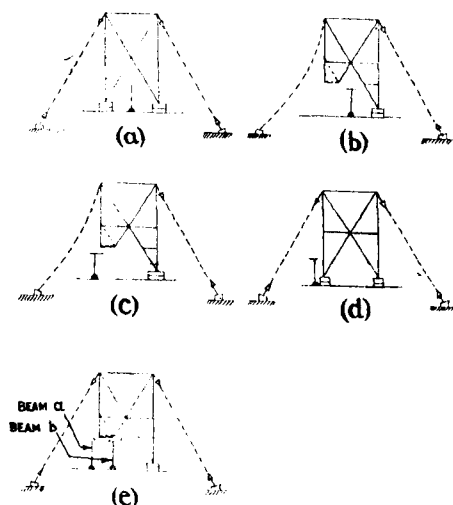


Fig. 34. Side shifting of a beam

used. The $\frac{1}{2}$ -in. rope from this winch is passed round the guide pulleys *d* and *e* in the nose of the launching girder, and then through a moving block hooked to the front crab I and a fixed

block at the tip of the mono-rail so as to give a velocity ratio of 3:1. This enhancement of the tractive force is necessary because of the small diameter (10 in.) of the wheels of the crab. Before starting the movement, the second crab and transport car II are tied together by means of a wire rope provided with a turn buckle in order to ensure direct transmission of the tractive effort from the crab to the car.

17.13.3. As the forward crabs I and II roll along the mono-rail bottom chord of the launching girder, stresses have a tendency to reach rather critical values in some of the top chord members of the launching girder. Although these values are by no means dangerous they are easily brought down by pumping water into the ballast tank which causes a negative moment in the bridge zone. As a matter of routine the ballast tank is filled to about 10 ft depth by the time the crab I approaches point 9 of the launching girder, and is emptied only after the crab passes point 14. This is repeated for every beam.

17.13.4. When the transport car II supporting the rear end of the beam approaches transport car I standing close to the standard 7, Fig. 32 (e), of the launching girder, the beam is stopped and connected to the crab III through the pair of holes *E* in the top flange, Fig. 33. Transport car II is then disengaged by disconnecting and lifting up the suspension rods. After connecting up crabs II and III by a wire rope tightened by turn buckle the forward movement of the beam is resumed. The beam is stopped when the bearing point *G* is exactly above the centre line of the pier, Fig 32 (f).

17.14. Side Shifting of Beam and Lowering it on Bearings

17.14.1. The beam is set down on the "shifting box" over the pier by releasing the jacks of crabs I and II. While the beam is still suspended from the rear crab III, the tail-piece, Fig. 27, is connected and the load transferred through it to the shifting beam by releasing the jacks in the rear crab. The three crabs are then disengaged by lifting up the suspension rods. The hinged part of the support 15/16 on the appropriate side is folded up and clamped in that position to permit the beam to be shifted sideways. To do this, the guy rope on that side to which the beam is to be shifted is slackened and the support jacked up on that side slightly to release the packing underneath. The packing is removed making the hinged part hang loose. The end of the wire rope from the forward winch on the launching girder is tied to the hinged part which is then slowly folded up by turning the winch. After this the beam is pulled sideways and set down as described in para. 17.2.7. The hinged part is again lowered, packed up and the guy rope tightened.

17.14.2. The beams are launched in the sequence *a-b-d-c*. After *b* beam is launched, the hinged part of 15/16 on that side remains folded up, Photo 38. Hence support 15/16 is packed tight over this beam. Similarly, beams *d* and *c* are set in position. Fig. 34 shows the various stages of side shifting of a beam.

It takes from 3 to 5 days for a beam starting from the north abutment to reach its final position and about 3 weeks for completing work of one span.

Cost Details

	Rs
Cost per cu.ft. of concrete	11/8/-
Cost per running foot of bridge	590/-/-
Cost of superstructure, estimated	13.18 lakh
Cost of entire bridge, estimated	34.5 lakh
Cost of imported materials for superstructure	3.7 lakh

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Appendix 1

CALCULATIONS OF STRESSES AND ELONGATIONS

The method of calculating the prestressing data (jack force and cable elongation) is illustrated in the following example:

Beam Type II₁₋₁₁

The maximum stresses in concrete due to permanent loads i.e., prestress and dead loads are:

1,580 lb per sq. in. in beam

840 lb per sq. in. in cantilever

The area of section in running sections is 1,088 sq. in. and 1,316 sq. in. at the beginning of the cantilever.

Force per cable = 40,500 kg

Evaluation of T_i

In Beam:

No. of cables, 12

$$\Delta T'_1 = 9 \times \frac{12 \times 40,500}{1088 \times 645} = 6 \text{ kg per sq. mm.}$$

$$\Delta T''_1 = 9 \times \frac{1,580}{1,420} = 10 \text{ kg per sq. mm.}$$

In cantilever:

No. of cables, 12

$$\Delta T'_1 = 9 \times \frac{12 \times 40,500}{1,316 \times 645} = 5 \text{ kg per sq. mm.}$$

$$\Delta T''_1 = 9 \times \frac{840}{1,420} = 5 \text{ kg per sq. mm.}$$

Average value for the whole beam before injection:

$$\Delta T'_1 = \frac{1}{2} (6+5) = 5 \text{ kg per sq. mm.}$$

Between supports, after injection:

$$\Delta T_1 = \frac{1}{2} (5+10) = 7.5 \text{ kg per sq. mm.}$$

Therefore,

$$T_i = 97 + 7.5 = 104.5 \text{ kg per sq. mm.}$$

For cantilever, after injection:

$$T_1 = \frac{1}{2} (5+5) = 5 \text{ kg per sq. mm.}$$

Therefore

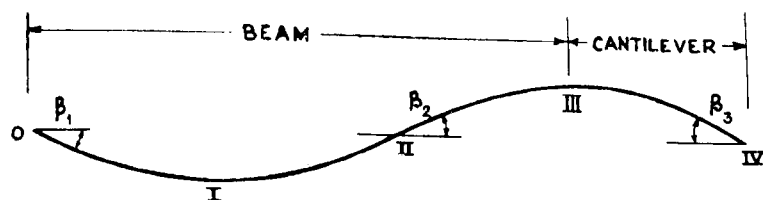
$$T_i = 97 + 5 = 102 \text{ kg per sq. mm.}$$

Elongation and Manometer Pressure

For illustration, take cable No. 3. The friction coefficients are worked out in Fig. 1 Appendix 1. Allowing 4 per cent for the loss in anchorage cones, we have

$$\text{on the left } T_o = 104.5 \times \frac{1}{.889} \times \frac{1}{.96} = 123 \text{ kg per sq. mm.}$$

$$\text{on the right } T_o = 102 \times \frac{1}{.905} \times \frac{1}{.96} = 118 \text{ kg per sq. mm.}$$



Length	Ft	51.7	44.7	32.1	33.3
Length	Meter	15.8	13.5	9.8	10.2
Angle	Degrees	12	10	10	13.5
0.4 (l+d)	per cent	11.1	9.4	7.9	9.5
T in kg per sq. mm.		104.5		102	

Fig. 1. Appendix 1

Adopt 123 kg per sq. mm., being the higher value, for both ends of the cable.

The minimum stress is at some point II' between I and III. Since this is the point of balance, it may be written down.

$$\begin{aligned} \Delta T_{o-II'} &= \Delta T_{IV-II'} \\ &= \frac{1}{2}(11.1 + 9.4 + 7.9 + 9.5) \\ &= 18.95 \text{ per cent} \end{aligned}$$

$$\Delta T_{I-II'} = 18.95 - 11.1 = 7.9 \text{ per cent}$$

$$\Delta T_{III-II'} = 18.95 - 9.5 = 9.4 \text{ per cent}$$

$$\Delta T_{min.} = 104.5 \times 0.921 = 96 \text{ kg per sq. mm.}$$

$$\Delta T_m = \frac{2 \times 96 + 123}{3} = 105 \text{ kg per sq. mm.}$$

From the stress-strain diagram for the steel, the elongation corresponding to a mean stress of 105 kg per sq. mm. is 5.10 per thousand.

$$\text{Cable length in beam} = 49.3 \text{ meters}$$

$$\begin{aligned} \text{Cable length between cone} \\ \text{face and wedge grips } 2 \times 0.4 &= 0.8 \text{ meters} \end{aligned}$$

$$\text{Total} = 50.1 \text{ meters}$$

$$\text{Total elongation } \frac{50,100 \times 5.10}{1,000} = 255 \text{ mm.}$$

$$\text{Total force in jack } 123 \times 461 = 56,700 \text{ kg}$$

$$\text{Manometer pressure } \frac{56,700}{160.6} = 353 \text{ kg per sq. mm.}$$

Appendix 2

LIST OF TOOLS AND PLANT

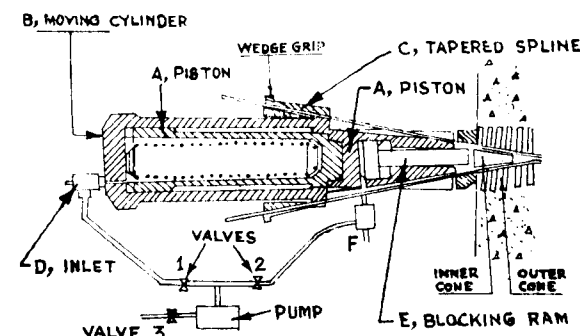
		No.
(a) Cable Yard		
Cable assembling machine		1
Sheath making machine		1
Allen-Randall cutter No. 4		1
Winch, 2-ton		1
Single sheave pulley, 6 in. diameter		1
Wire rope, $\frac{1}{4}$ in. diameter	400 rft.	
(b) Casting Yard		
Steel forms	1 set	
Gantry with 2 chain hoists, each of $1\frac{1}{2}$ -ton capacity	1 No.	
Concrete mixers, 7/10	3	
Platform scale, 1 ton	1	
Weigh batcher, double bucket	1	
Electric vibrators, 1,500 watts	4	
Electric vibrators, 850 watts	6	
Needle vibrator, compressed air	1	
Transformers, 6,000 watts	1	
Welding machine, 20-300 amps.	1	
Transformers, 3,000 watts	2	
Water pump, 4 in. by 3 in. electric, 5 H.P.	1	
(c) Moving Beams in Production Yard		
Winch, 10-ton	2	
Winch, 2-ton	3	
Wire rope, $7/8$ in. diameter	1,000 rft	
3-sheave pulley, 9 in. diameter	1 No.	
2-sheave pulley, 9 in. diameter	2	
Single pulley, 9 in. diameter	2	
Hydraulic jack, 200 ton	1	
Hydraulic jack, 45 ton	1	
(d) Prestressing and Grouting		
Hydraulic pump, 5000 lb per sq. in.		1
Trolleys	(2 × 4)	8
Freyssinet jack $12/7$ mm with standard accessories and tools and pump fitted with calibrated manometer and hose connections		2
Wooden spreader disc, about 12 in. diameter, with 3-in. diameter hole in centre and 12 notches on periphery		2
Wooden strut $2\frac{1}{2}$ ft by $1\frac{1}{2}$ in. diameter		
Soft iron pin-hammer		2
Medium sledge-hammer		2
Scale marked in mm with a handle		2
Chalk pieces, crayons, etc.		
Dial deflectometers reading to $1/2000$ in.		6
Screen 10 ft × 10 ft made of bamboo matting reinforced with closely spaced bamboos		2
Red flags		2
Light site-made crane		2
Crane, $\frac{1}{2}$ ton		2
Prestressing pump, 600 kg per sq. cm.		2
Oxy-acetylene gas :		
Cutter		1
Bracing unit		1
Gas cylinder		
Oxygen		2
Acetylene		2
Colcrete mixer with 50 ft of $\frac{3}{4}$ in. diameter hose and nozzle		1
Air compressor, electric, 22 cu. ft. at 125 lb per sq. in.		1
(e) Launching		
Launching girder		1
Transport cars		2

	No.
Shifting beam	1
Tail piece	1
Shifting box	1
Sliding plate	1
Roller grids, long	3
Roller grids, short	3
Travelling crabs	3
Hangers, 70 mm	4
Hangers, 56 mm	6
Hydraulic jacks, 25 ton	6
Hydraulic jack 50 tons	3
Hydraulic jacks, 200 ton	1
Hydraulic dual pumps, 5000 lb per sq. in.	3
4-way connection	1
2-way connection	2
Hydraulic pump	1
Wire rope, $\frac{1}{2}$ in. diameter	1,200 rft
Winch, 2 ton	3 No.
3-sheave pulley, 4 in. diameter	3
2-sheave pulley, 4 in. diameter	3
Single-sheave pulley, 6 in. diameter	4
Manila rope, $\frac{5}{8}$ in. diameter	1,000 rft
Chain hoists, $1\frac{1}{2}$ ton	3 No.
Water pump, electric, 5 H.P., 4 in. by 3 in.	1
(f) Laboratory Equipment	
Table vibrator, 3 H. P.	1
Compression testing machine, 150 ton	1
Laboratory van equipped for concrete testing	1
6-in. cube mould (4 cube)	1
6-in. cube mould (1 cube)	1

Appendix 3

DESCRIPTION OF FREYSSINET JACK

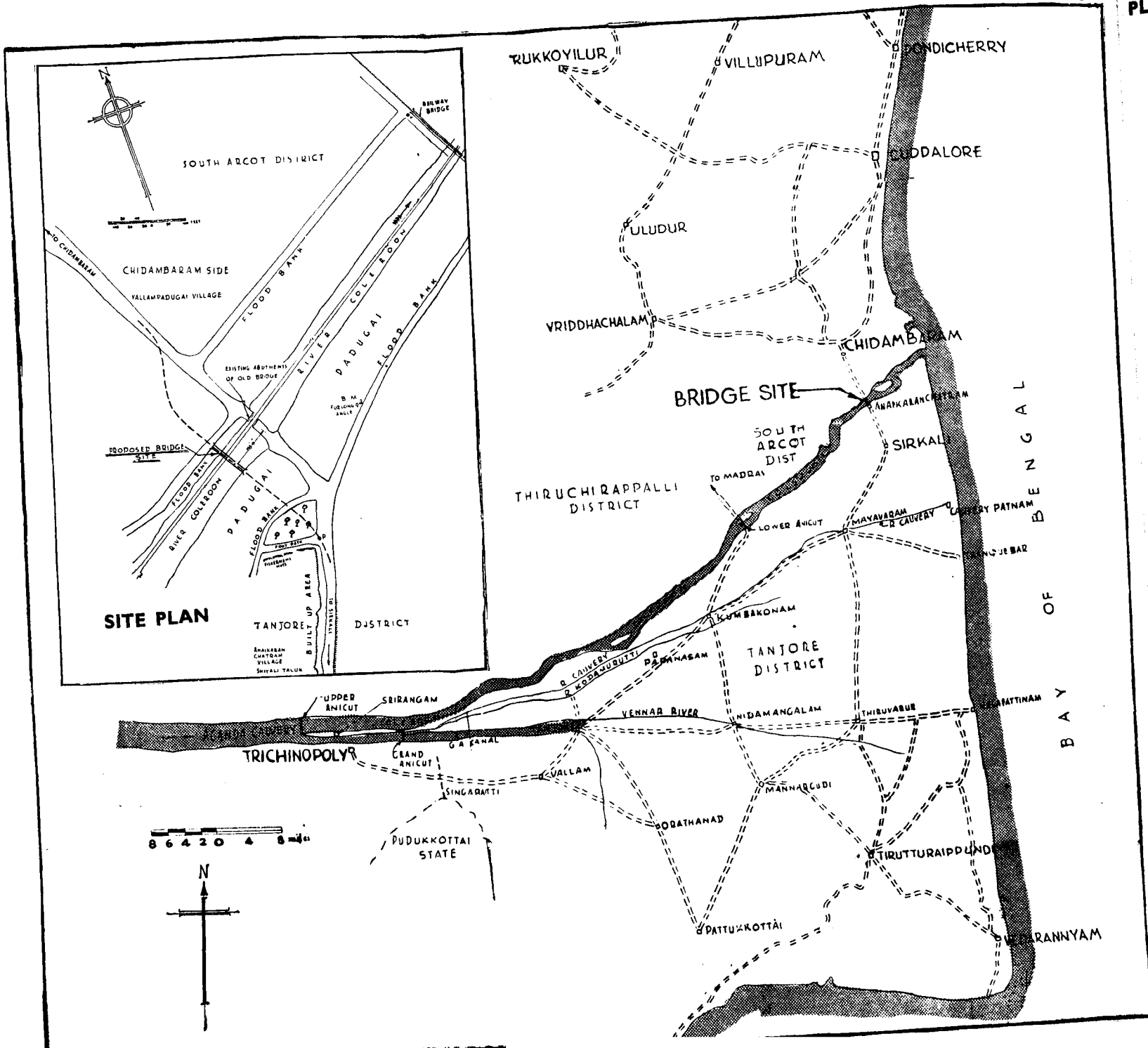
Fig. below shows a longitudinal section of the jack. A piston assembly, *A*, bears on the outer cone through a grooved foot and a self centering collar. The piston carries a moving cylinder, *B*, incorporating six tapered splines, *C*, at the near end. The cable wires are passed through the grooves in the foot and wedged between the splines on the cylinder. Water or oil is then pumped in through the inlet, *D*, causing the cylinder to move out stretching the wires. When the required extension has been reached, the blocking ram *E*, is actuated by hydraulic pressure through inlet, *F*, and this forces the inner cone into the anchorage. Internal helix springs are provided to return the cylinder and ram on release of the hydraulic pressure.



Freyssinet Jack 1951 Model

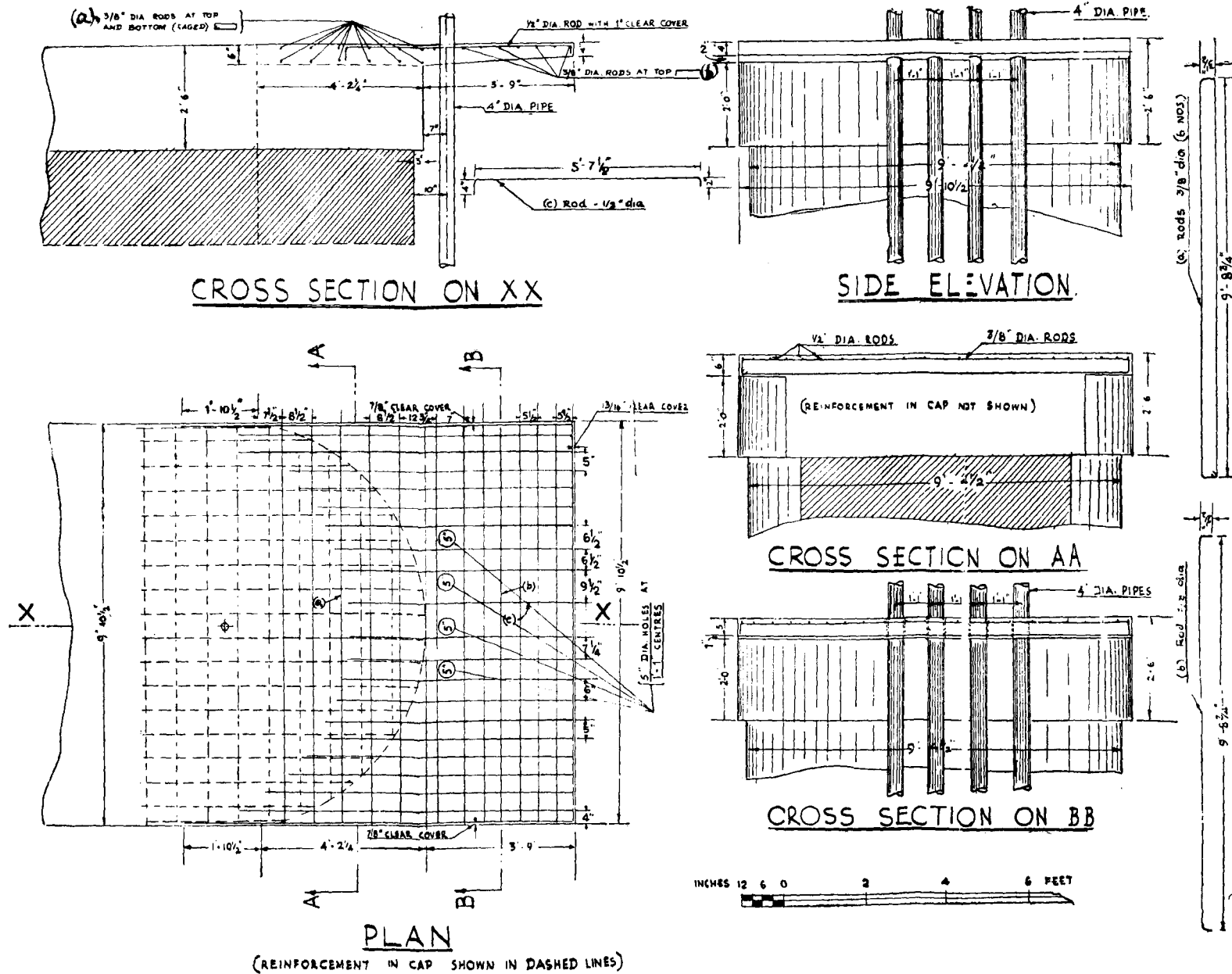
The principal details of the jack are summarised below. Photos 25 and 26 show general views of a jack.

Overall length cloud	92 cm	36 in.
Travel	300 mm	$11\frac{1}{4}$ in.
Tensioning piston		
Diameter	14.3 cm	5.64 in.
Area	160.6 sq. cm.	24.85 sq. in.
Blocking ram		
Diameter	7.9 cm	3.12 in.
Area	49.4 sq. cm.	7.63 sq. in.
Maximum tensioning force	73,000 kg	161,000 lb
Total weight	85 kg	187 lb



LOCATION OF COLEROON BRIDGE

PLATE III



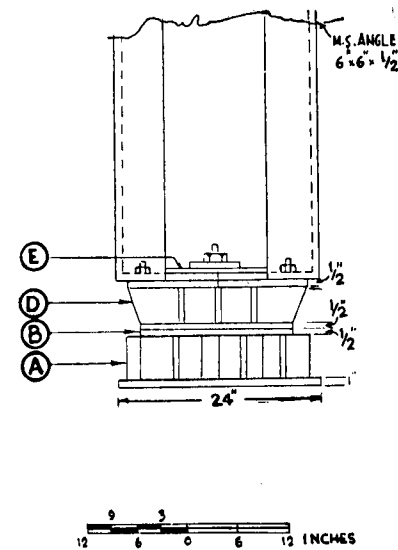
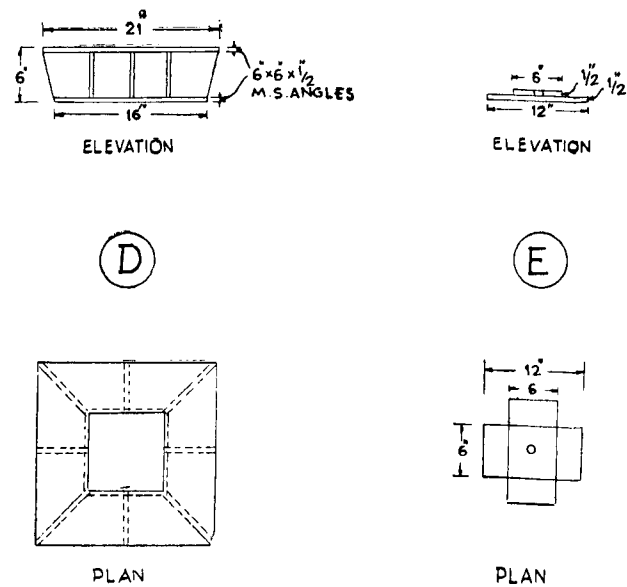
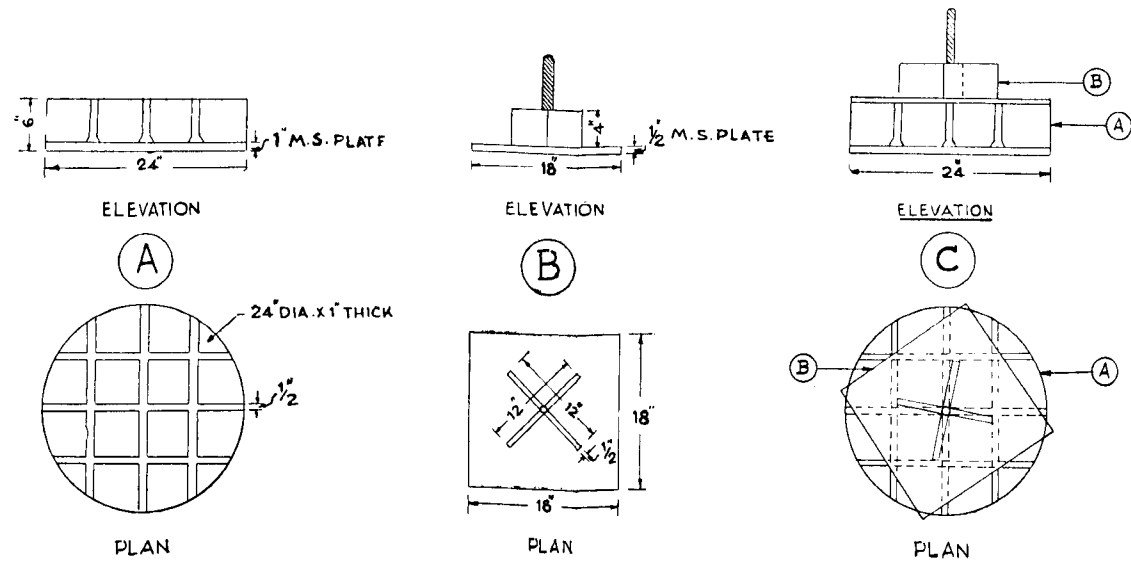
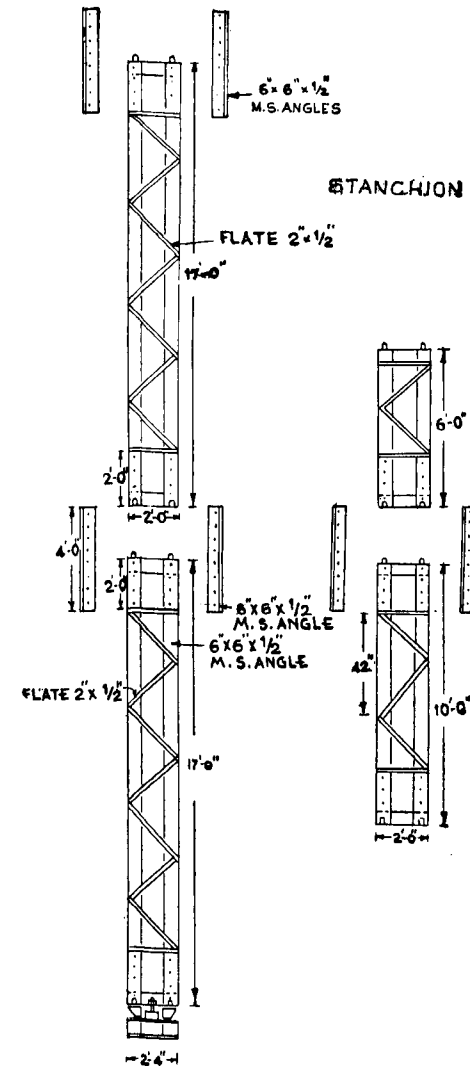
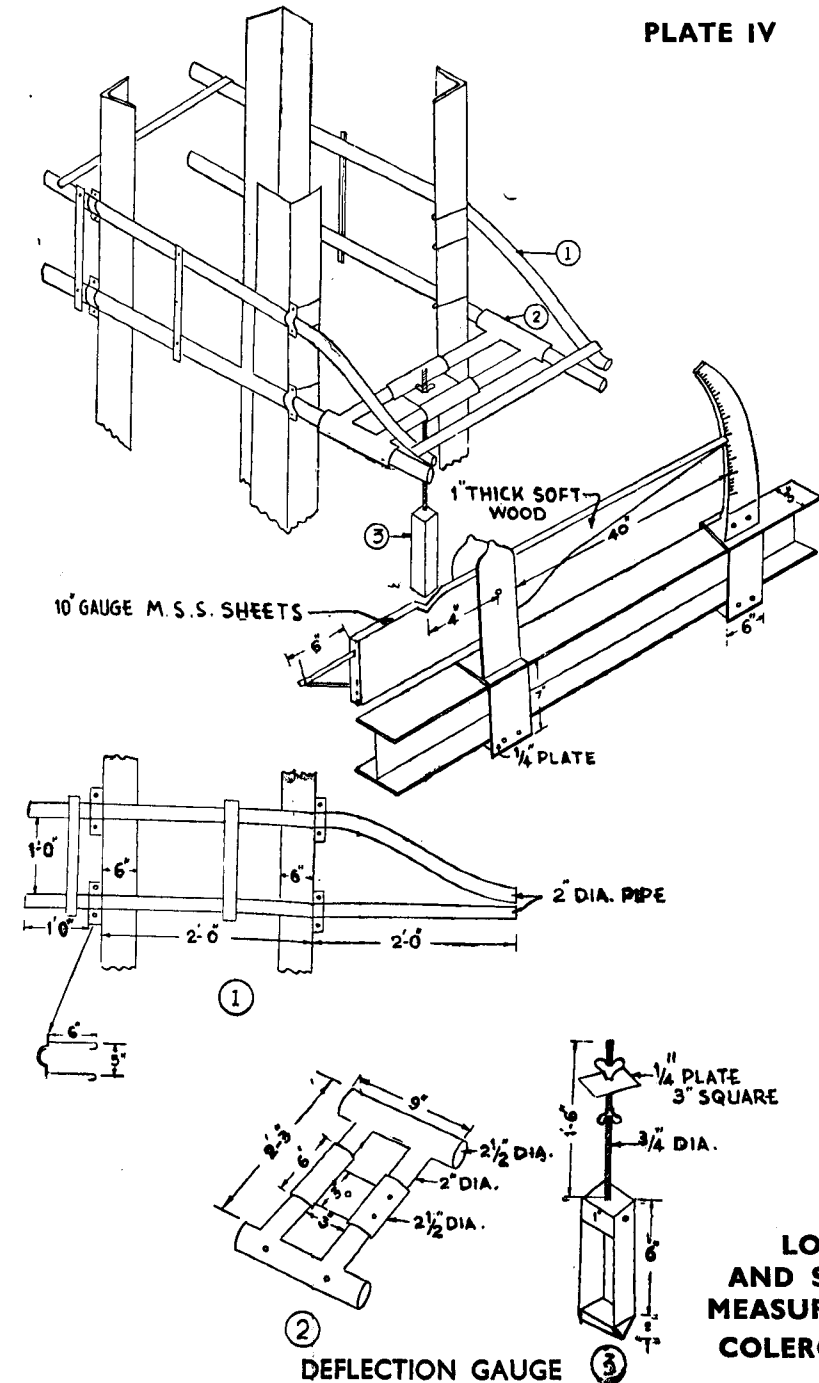
BEARING PLATE ASSEMBLY
FOR LOAD TESTSTANCHION FOR THE
LOAD TEST

PLATE IV

LOAD TEST
AND SETTLEMENT
MEASURING DEVICES,
COLEROON BRIDGE

Paper No. 193

**"INFLUENCE FIELDS FOR FINITE CONCRETE
SLABS SUPPORTED FREELY ALONG TWO
OPPOSITE EDGES AND FREE AT THE
OTHER TWO"***

By

K. C. RAY,

R. SUBRAMONIAM, and

S. BADARUDDIN

SYNOPSIS

The Paper presents unit influence fields for R. C. slabs freely supported on two sides and having breadth span ratios of 1, 1.2, 1.5, 2.0, 3.0 and 6.0. A worked out example for two lanes of I.R.C. Class A loading and one lane of I. R. C. Class AA loading is given.

1. INTRODUCTION

Reinforced concrete slabs, which are used for spans upto 30 ft, generally rest freely on the supports. Analysis of such slabs, considering them as plates, had been made by Westergaard on the assumption that the plate was infinitely long. Westergaard pointed out that values obtained for infinitely long slabs would not vary much from those for slabs finite in length. Adolf Pucher of Germany had published influence fields for finite slabs, but the span-breadth ratio adopted by him did not cover the cases met with in practice. Also, they did not give values for bending moments directly under the load. So, values for almost all the cases which will arise in practice and a method of finding out the co-efficients for bending moments under the load have been worked out.

* Guided by Prof. Gerald Pickett.

This analysis had been based upon the plate theory. The only assumption made is that the Poisson's ratio for reinforced concrete is zero.

Results have been presented in a graphical form for ready use,

2. NOTATIONS

D = Flexural rigidity of the slab.

$$= \frac{Eh^3}{12(1-\mu^2)}$$

X, Y = Rectangular co-ordinates of points for which moments are required.

X_0, Y_0 = Rectangular co-ordinates of points of application of concentrated load.

3. DERIVATIONS

3.1. Assumptions

- (1) Slab is homogeneous, isotropic, elastic, thin and uniform in thickness.
- (2) Loads are applied in a direction normal to the surface of the slab.

3.2. Equations

$$D \nabla^2 \nabla^2 \omega = p \quad \text{----- (1)}$$

where ω = deflection +ve downwards

p = intensity of loading

Since

$$\omega = 0$$

$$M_x = -D \frac{\partial^2 \omega}{\partial x^2} \quad \text{----- (2)}$$

$$M_y = -D \frac{\partial^2 \omega}{\partial y^2} \quad \text{----- (3)}$$

$$M_{xy} = D \frac{\partial^2 \omega}{\partial x \partial y} \quad \text{----- (4)}$$

$$V_y = -D \left[\frac{\partial^3 \omega}{\partial y^3} + 2 \frac{\partial^3 \omega}{\partial y \partial x^2} \right] \quad \text{----- (5)}$$

$$V_x = -D \left[\frac{\partial^3 \omega}{\partial x^3} + 2 \frac{\partial^3 \omega}{\partial x \partial y^2} \right] \quad \text{----- (6)}$$

3.3. General Expressions for Moments

Consider a slab freely supported along edges AB and CD , Fig. 1. Let a concentrated load P act on the slab at the point X_0, Y_0 , with reference to the axes AX and AY .

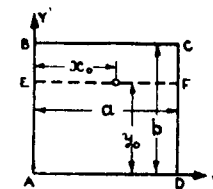


Fig. 1.

$$\text{Represent } p = \int_0^a f(x) dx \quad \text{----- (7)}$$

where $f(x)$ is the intensity of line loading on the line $y = y_0$ and can be represented generally by

$$f(x) = \sum_{n=1,2,3}^{\infty} k_n \sin\left(\frac{n\pi x}{a}\right) \quad \text{----- (8)}$$

By Fourier transformation

$$k_n = \frac{2P}{a} \sin\left(\frac{n\pi x_0}{a}\right) \quad \text{----- (9)}$$

Substituting, in (8)

$$f(x) = \frac{2P}{a} \sum_{n=1,2,3}^{\infty} \sin\left(\frac{n\pi x}{a}\right) \sin\left(\frac{n\pi x_0}{a}\right) \quad \text{----- (10)}$$

Slab is divided in two regions $AEFD$, where $y_0 \geq y$, and $EBCF$, where $y_0 < y$.

The differential equation for deflection can be represented for both the regions by,

$$D \nabla^2 \nabla^2 \omega = 0 \quad \text{excepting the common boundary } EF.$$

Solution for both the regions can be written as below :

For $A E F D$ -

$$\begin{aligned} \omega = & \sum_{n=0,1,2}^{\infty} \sin\left(\frac{n\pi x}{a}\right) \left[A_n \cosh\left(\frac{n\pi y}{a}\right) \right. \\ & + B_n \sinh\left(\frac{n\pi y}{a}\right) + C_n \left(\frac{n\pi y}{a}\right) \sinh\left(\frac{n\pi y}{a}\right) \\ & \left. + D_n \left(\frac{n\pi y}{a}\right) \cosh\left(\frac{n\pi y}{a}\right) \right] \quad \text{----- (11)} \end{aligned}$$

For $E B C F$ -

$$\begin{aligned} \omega' = & \sum_{n=0,1,2}^{\infty} \sin\left(\frac{n\pi x}{a}\right) \left[A'_n \cosh\left(\frac{n\pi y}{a}\right) \right. \\ & + B'_n \sinh\left(\frac{n\pi y}{a}\right) + C'_n \left(\frac{n\pi y}{a}\right) \sinh\left(\frac{n\pi y}{a}\right) \\ & \left. + D'_n \left(\frac{n\pi y}{a}\right) \cosh\left(\frac{n\pi y}{a}\right) \right] \quad \text{----- (12)} \end{aligned}$$

where A_n, B_n, C_n, D_n and $A'_n, B'_n, \dots, D'_n$ are constants.

Boundary conditions :

$$M_y|_{y=0} = 0 \quad \text{----- (13)}$$

$$V_y|_{y=0} = 0 \quad \text{----- (14)}$$

$$M_y|_{y=b} = 0 \quad \text{----- (15)}$$

$$V_y|_{y=b} = 0 \quad \text{----- (16)}$$

From continuity conditions at the common boundary, the following relationships are obtained.

$$M_y|_{y=y_0} = M_y|_{y=y_0} \quad \text{----- (17)}$$

$$V_y|_{y=y_0} = V_y|_{y=y_0} \quad \text{----- (18)}$$

$$\omega|_{y=y_0} = \omega|_{y=y_0} \quad \text{----- (19)}$$

$$\frac{\partial \omega}{\partial y}|_{y=y_0} = \frac{\partial \omega}{\partial y}|_{y=y_0} \quad \text{----- (20)}$$

From above eight boundary conditions, equations (13) to (20) the eight constants involved in equations (11) and (12) have been determined as follows :

$$C_n = \frac{-\alpha(\phi_1 + \phi_2) - 3\phi_2 \tanh \alpha}{\alpha^2 (\coth \alpha - \tanh \alpha) - 9 \tanh \alpha} \quad \text{----- (21)}$$

$$D_n = \frac{\phi_2 \alpha \tanh \alpha - (3 - \alpha \coth \alpha) \phi_1}{\alpha^2 (\coth \alpha - \tanh \alpha) - 9 \tanh \alpha} \quad \text{----- (22)}$$

$$A_n = 2C_n \quad \text{----- (23)}$$

$$B_n = D_n \quad \text{----- (24)}$$

Where,

$$\phi_1 = K [1 - \tanh \alpha \coth \beta + (\alpha - \beta)(\tanh \alpha - \coth \beta)] \sinh \beta \quad \text{----- (25)}$$

$$\phi_2 = K [2(1 - \coth \alpha \coth \beta) + (\alpha - \beta)(\coth \beta - \coth \alpha)] \sinh \beta \quad \text{----- (26)}$$

$$\alpha = \frac{n\pi b}{a}, \quad \beta = \frac{n\pi y_0}{a}; \quad K = \frac{Pa^3}{(n\pi)^3 D} \sin\left(\frac{n\pi x_0}{a}\right) \quad \text{----- (27)}$$

Values for A'_n, B'_n etc. are not given, as they would not be necessary while working a particular case, as equation (11) would be ample to find out the moments etc. for a particular point which, in the case under consideration, are the centre of the slab and the centre of the free edge.

3.4. Particular Case

General expressions for bending moments are given by :

$$M_x = -D \frac{\partial^2 \omega}{\partial x^2}, \quad M_y = -D \frac{\partial^2 \omega}{\partial y^2}$$

Differentiating the expression for ω in equation (11) we get :

$$M_x = -D \sum \left(\frac{n\pi}{a} \right)^2 \sin \frac{n\pi x}{a} \left[A_n \cosh \left(\frac{n\pi y}{a} \right) + B_n \sinh \left(\frac{n\pi y}{a} \right) + C_n \left(\frac{n\pi y}{a} \right) \sinh \left(\frac{n\pi y}{a} \right) + D_n \left(\frac{n\pi y}{a} \right) \cosh \left(\frac{n\pi y}{a} \right) \right] \quad (28)$$

$$M_y = -D \sum \left(\frac{n\pi}{a} \right)^2 \sin \frac{n\pi x}{a} \left[3D_n \sinh \left(\frac{n\pi y}{a} \right) + C_n \left(\frac{n\pi y}{a} \right) \sinh \left(\frac{n\pi y}{a} \right) + D_n \left(\frac{n\pi y}{a} \right) \cosh \left(\frac{n\pi y}{a} \right) \right] \quad (29)$$

For the centre of the slab where co-ordinates are $\frac{a}{2}, \frac{b}{2}$, taking $K = \frac{Pa^2}{(n\pi)^3 D} \sin \frac{n\pi x}{a}$ we get:

$$M_x = P \sum_{n=1,3,5}^{\infty} \frac{n^{\frac{1}{2}}}{(-1)^{\frac{n-1}{2}}} \frac{\sin \left(\frac{n\pi x}{a} \right)}{n\pi} \left[\left(\frac{Dn}{K} + \frac{C}{2} \cdot \frac{Cn}{K} \right) \sinh \frac{C}{2} + \left(\frac{C}{n} \cdot \frac{Dn}{K} - \frac{2Cn}{K} \right) \cosh \frac{C}{2} \right] \quad (30)$$

$$M_y = P \sum_{n=1,3,5}^{\infty} \frac{n^{\frac{1}{2}}}{(-1)^{\frac{n-1}{2}}} \frac{\sin \left(\frac{n\pi x}{a} \right)}{n\pi} \left[\left(\frac{3Dn}{K} + \frac{C}{2} \cdot \frac{Cn}{K} \right) \sinh \frac{C}{2} + \frac{C}{2} \cdot \frac{Dn}{K} \cosh \frac{C}{2} \right] \quad (31)$$

Similarly, for the centre of the free edge, where co-ordinates are $\frac{a}{2}, 0$, we get,

$$M_x = P \sum_{n=1,3,5}^{\infty} \frac{n^{\frac{1}{2}}}{(-1)^{\frac{n-1}{2}}} \frac{\sin \left(\frac{n\pi x}{a} \right)}{n\pi} \frac{2Cn}{K} \quad (32)$$

$$M_y = 0$$

Values for M_x and M_y have been evaluated taking $P = 1.00$ for the following particular cases:

$$\frac{b}{a} = 1.00; 1.20; 1.50; 2.00; 3.00; \text{ and } 6.00$$

Thus unit influence fields have been arrived at. The variation of co-efficients for moments at very close distances from the point of application of the load when it is acting on the points for which moment are required have been considered. An extrapolation curve has been established to find the near correct value for moments when the load is acting on the point for which the values are sought for.

Influence fields for M_x and M_y at the centre of the slab and M_x at the centre of the free edge for the various breadth-span ratios mentioned in para. 3.4 are given in Plates I, II and III respectively.

4. NUMERICAL EXAMPLE

Design a freely supported R. C. slab for a bridge which is 30 ft wide with the following data:

Effective span of 20 ft

Loading - 2 lanes of Indian Roads Congress Class A or a single lane of Indian Roads Congress Class AA whichever produces greater stresses.

$F_o = 1,350$ lb per sq. in. in bending;

$F_s = 18,000$ lb per sq. in.

$m = 10.$

Dead Load

Assume a slab 15 in. thick :	lb per sq. ft.
weight	= 180
4-in. macadam	= 50
1-in. asphalt	= 10
Total	= 240

Dead Load Moments

$$\begin{aligned} \text{At the centre of span } (M_x) &= \frac{wl^2}{8} = \frac{240 \times 20 \times 20}{8} \\ &= 12,000 \text{ ft-lb per ft} \\ &= 12.00 \text{ kip-ft per ft} \\ (M_y) &= 0.00 \\ \text{At the centre of free edge } (M_x) &= 12.00 \text{ ft-kips per ft} \\ (M_y) &= 0.00, \end{aligned}$$

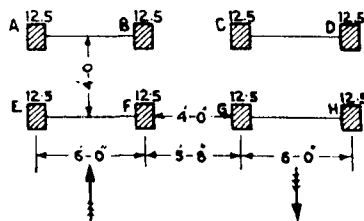
Live Load

Impact factor for Class A for a 20 ft span = 0.41

-do- Class AA -do- = 0.10.

Moments due to Class —Two Lanes

Loading system is as follows :



ig. 2.

From dimensions it is very clear that for a 20 ft span, loads A—H will only be operative for worst effects.

The loads are plotted on a piece of tracing paper and oriented in such a manner over the appropriate influence field as to yield maximum moment.

Considering the loads as concentrated loads, values of moments with impact are found as under :

+ve maximum M_x at centre =

$$1.41 \times 12.5 (0.11 + 0.88 + 0.12 + 0.17 + 0.11 + 0.38 + 0.11 + 0.17) = 22.03 \text{ ft kips per ft}$$

-ve maximum M_x at centre = 0

$$+ve \text{ maximum } M_y \text{ at centre} = 1.41 \times 12.5 (0.35 + 0.09 - 0.02 - 0.005)$$

$$= 7.32 \text{ ft kips per ft}$$

$$-ve \text{ maximum } M_y \text{ at centre} = 1.41 \times 12.5 (0.06 + 0.037 + 0.025 + 0.01)$$

$$= 2.15 \text{ ft kips per ft}$$

+ve maximum M_x at the centre of free edge

$$= 1.41 \times 12.5 (0.7 + 0.24 + 0.025 + 0.17 + 0.115 + 0.08 + 0.06 + 0.04) = 28.72 \text{ ft kips per ft}$$

-ve maximum M_x at the centre of free edge = 0.00

M_y at the centre of free edge = 0.00

Moments due to Class AA Single Lane

Loading is as shown below :

Contact Area

Load intensity per sq. ft. of contact area with impact

$$= \frac{1.1 \times 70 \times 2240}{12 \times 10} = 1.437 \text{ kips per sq. ft.}$$

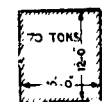


Fig. 3.

At the centre of the slab :

The load is applied over the central area and the volume of the field is calculated as per Simpson's rule. This volume is multiplied by the load intensity to give the moment.

In a more approximate form, this distributed load could have been split up into small concentrated loads and factors read directly. The former method is chosen to give more accurate results.

$$\text{Maximum } +ve M_x = 17.86 \times 1.437 = 25.66 \text{ ft kips per ft}$$

$$\text{Maximum } +ve M_y = 6.98 \times 1.44 = 10.04 \text{ ft kips per ft}$$

$$\text{Maximum } -ve M_x = 0.00$$

$$\text{Maximum } -ve M_y = 2.53 \times 1.437 = 3.64 \text{ ft kip per ft}$$

At the centre of the free edge :

$$\text{Maximum } +ve M_x = 25.92 \times 1.437 = 37.25 \text{ ft kip per ft}$$

$$M_y = 0.00$$

TABLE OF MOMENTS

	Dead load Moment M_x ft-kip per ft	Live load moments M_x in ft-kips per ft					
		+ve Maximum			-ve Maximum		
		Class A	Class AA	Combined Maximum	Class A	Class AA	Combined Maximum
Centre of slab	12.0	22.03	25.66	37.66	0	0	0
Centre of free edge	12.0	28.72	37.25	49.25	0	0	0

	Dead load Moment ft kip per ft	Live load moments M_y in ft kip per ft					
		+ve Maximum			-ve Maximum		
		Class A	Class AA	Combined Maximum	Class A	Class AA	Combined Maximum
Centre of slab	0	7.32	10.04	10.04	2.15	3.64	3.64
Centre of free edge	0	0	0	0	0	0	0

Design

The maximum +ve B.M., M_x at the centre of free edge
= 49.25 ft kip per ft

Equating the B.M. to moment of resistance of the slab

$$Qbd^2 = \text{B.M.}$$

$$248 \times 12 \times d^2 = 49.25 \times 1000 \times 12 \text{ in ft-lb}$$

$$\therefore d = 14.0 \text{ in.}$$

Since the total depth of the slab is assumed as 15 in., there is ample cover.

$$\begin{aligned} \text{Area of steel required} = A_s &= \frac{M}{j_d f_s} \\ &= \frac{49.25 \times 1000 \times 12}{.857 \times 14 \times 18000} \\ &= 2.74 \text{ sq. in. per ft} \end{aligned}$$

The maximum +ve M_x at the centre of the slab
= 37.66 ft kip per ft

$$A_s = \frac{37.66 \times 1000 \times 12}{.857 \times 14 \times 18000} = 2.00 \text{ sq. in. per ft}$$

-ve moment M_x in the slab is zero and so nominal reinforcement may be provided.

Maximum +ve moment M_y at the centre of the slab = 10.04 ft-kip per ft

$$A_s = \frac{10.04 \times 1000 \times 12}{0.857 \times 13.0 \times 18000} = 0.60 \text{ sq. in. per ft}$$

Maximum -ve moment M_y at the centre of the slab = 3.64 ft kip per ft

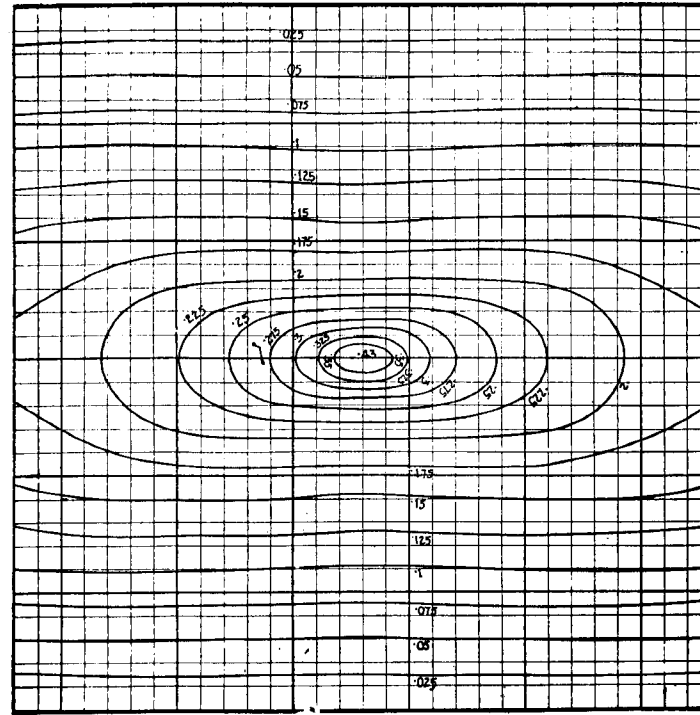
$$A_s = \frac{3.64 \times 12 \times 1000}{0.857 \times 18000 \times 13} = 0.22 \text{ sq. in. per ft}$$

It may be noted here that the maximum M_{xy} at the centre of the free edge works out to

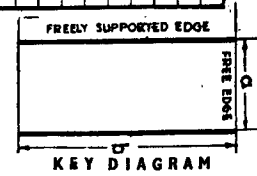
$$\begin{aligned} M_{xy} &= \frac{M_x - M_y}{2} = \frac{M_x}{2} \text{ as } M_y = 0 \\ &= \frac{49.25}{2} = 24.7 \text{ ft-kip per ft} \end{aligned}$$

$$\text{Force per ft width diagonally} = \frac{24.7 \times 12 \times 1000}{2/3 \times 14} \text{ lb}$$

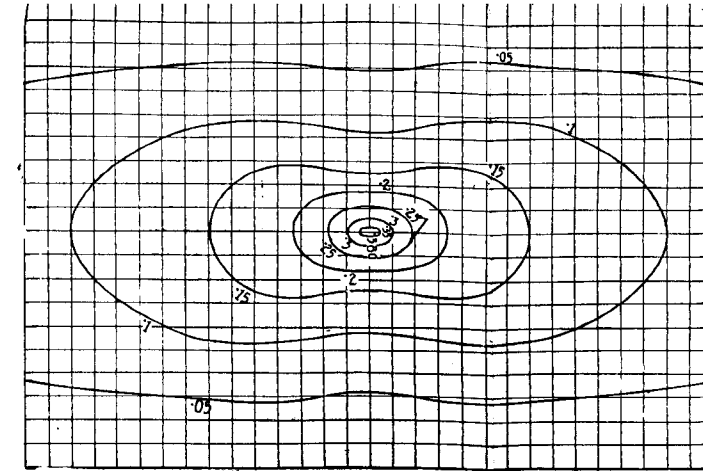
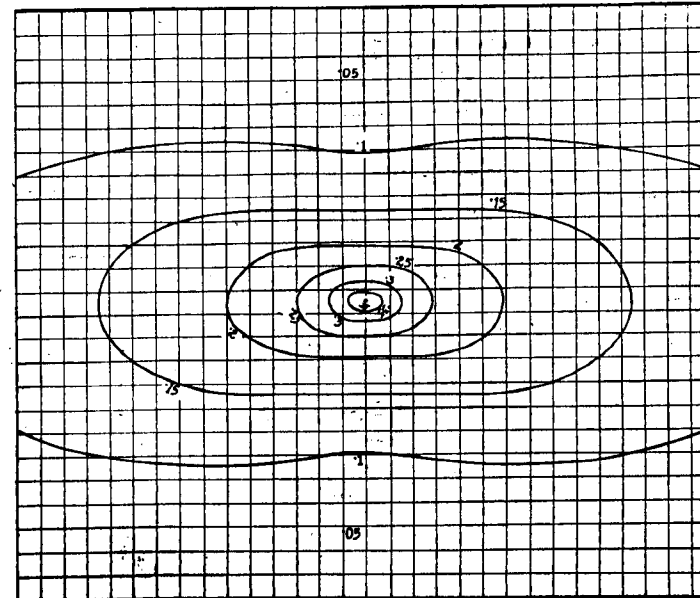
$$\begin{aligned} A_s &= \frac{24.7 \times 12 \times 1000 \times 3}{2 \times 14 \times 18000} \\ &= 1.77 \text{ sq. in. per ft} \end{aligned}$$



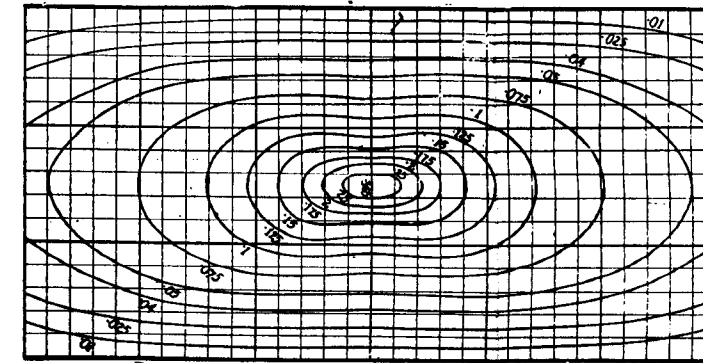
$b/a = 1.00$



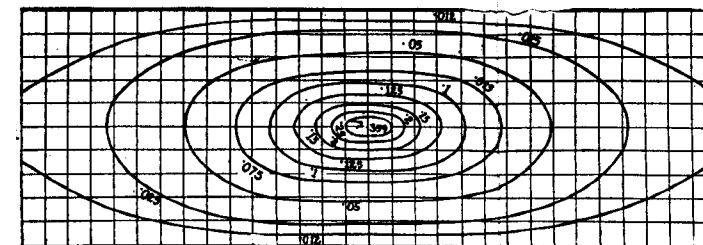
Note:—Moment Units=Load Unit $\times$ Linear Unit per Linear Unit;
 Moment=Coefficient $\times$ Load (for point loads);
 Moment=Contour Coefficient Volume $\times$ Load intensity (for distribution loads)



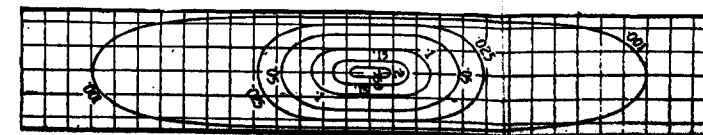
$b/a = 1.50$



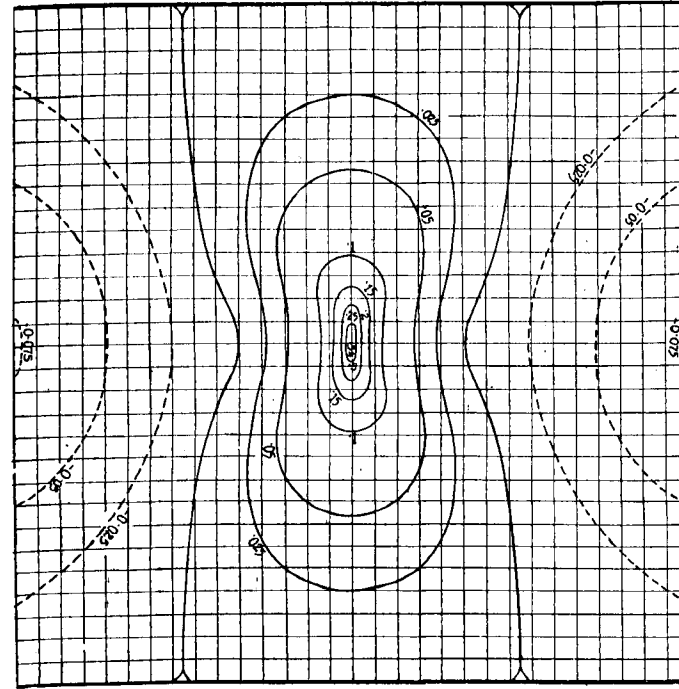
$b/a = 2.00$



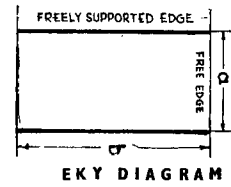
$b/a = 3.00$



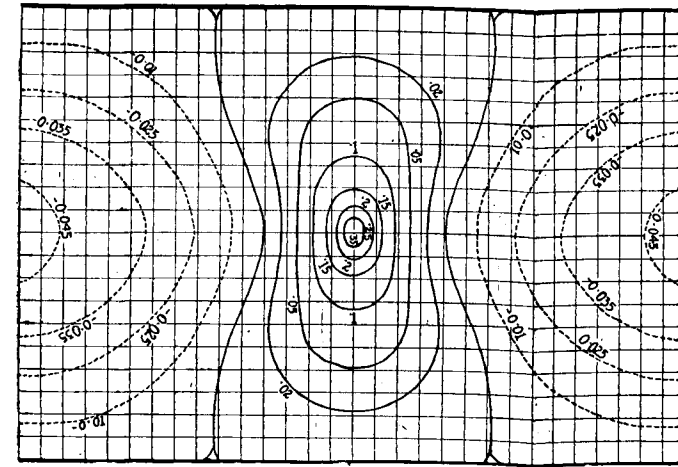
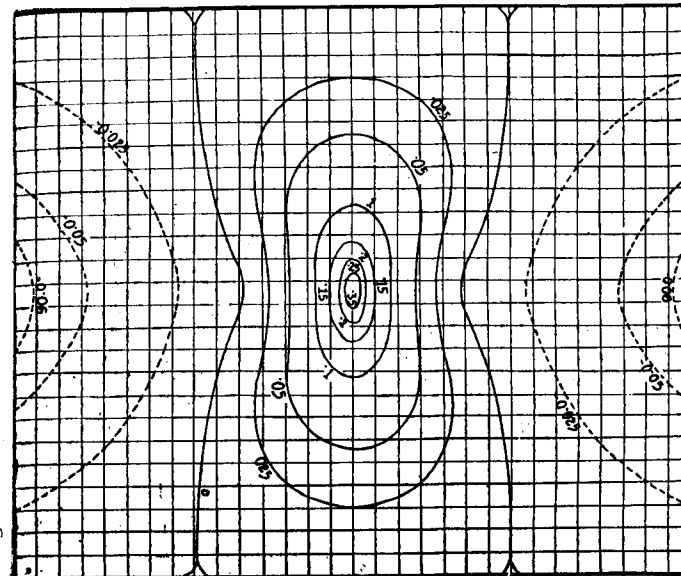
**MOMENT COEFFICIENT
 CONTOURS
 M_x AT CENTRE OF SLAB**



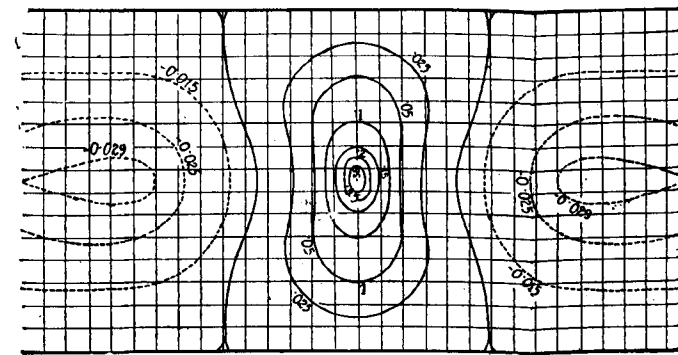
$b/a = 1.00$



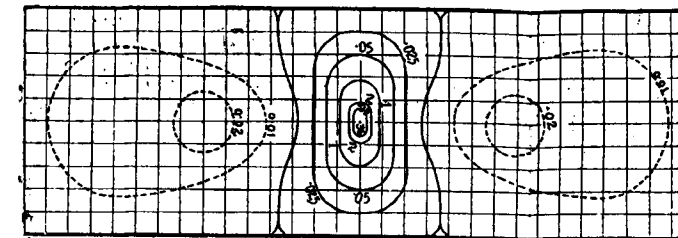
Note:—Moment Units=Load Unit $\times$ Linear Unit per linear unit;
 Moment=Coefficient $\times$ Load (for point loads); Moment=Con-
 tour Coefficient Volume $\times$ Load intensity (for distribution
 loads)



$b/a = 1.50$



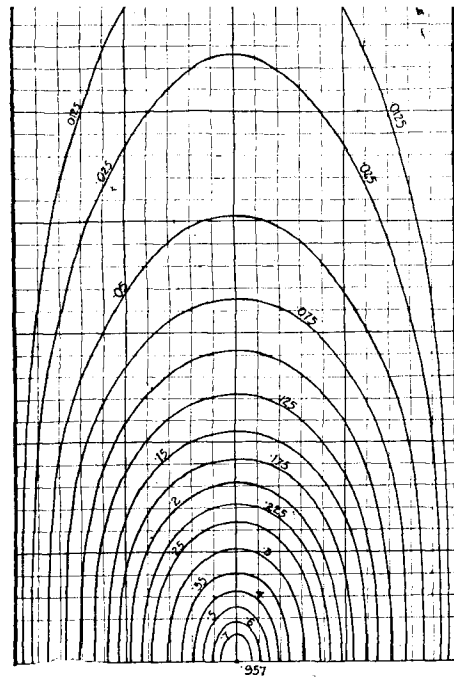
$b/a = 2.00$



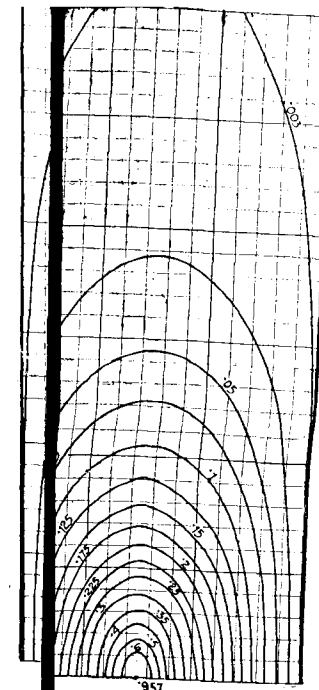
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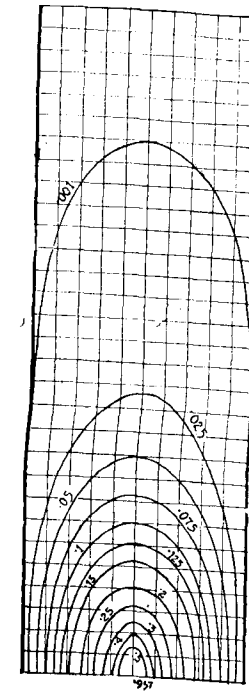
**MOMENT COEFFICIENT
 CONTOURS
 M_y AT CENTRE OF SLAB**



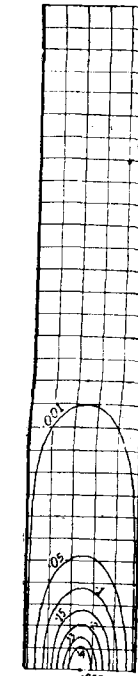
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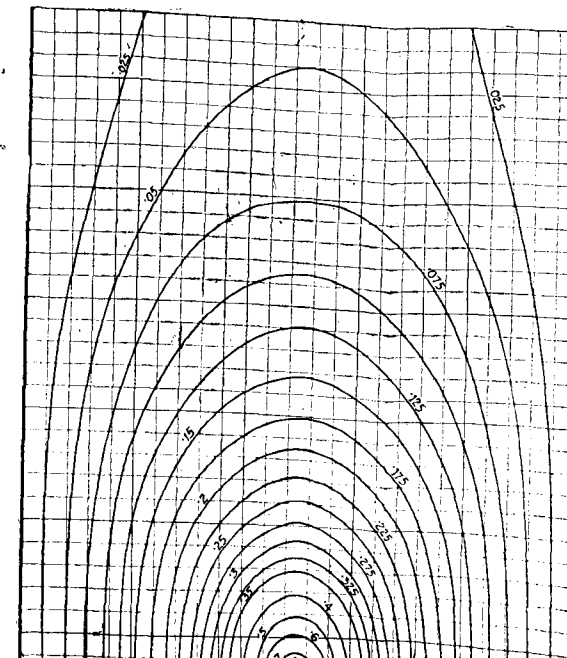
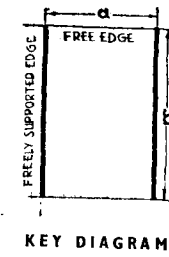
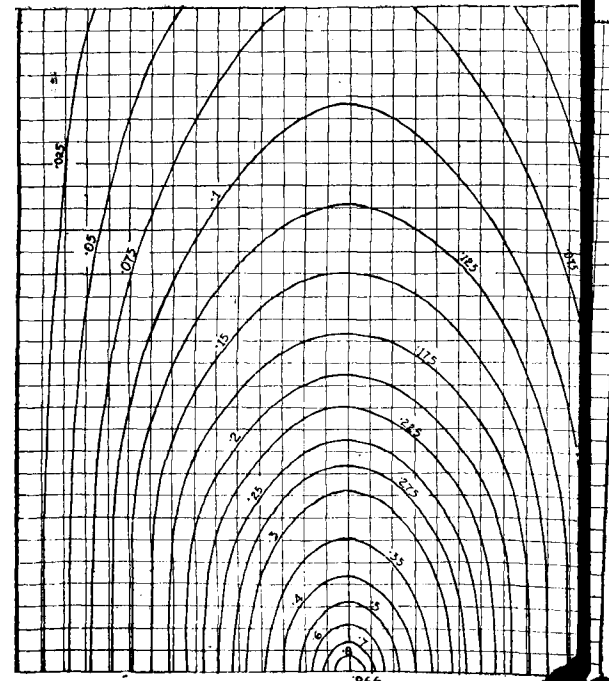
$b/a = 2.0$



$b/a = 3.0$



$b/a = 6.0$



Note :—Moment Units=Load Unit $\times$ Linear Unit per linear Unit ;
 Moment=Coefficient $\times$ Load (for point loads);
 Moment=Contour Coefficient Volume $\times$ Load Intensity
 (for distribution loads)

MOMENT CO-EFFICIENT
 CONTOURS
 M_x AT CENTRE OF FREE
 EDGE

INFORMATION SECTION

SURFACE DRESSING PRACTICE IN VICTORIA

By

HARI DUTT GUPTA

CONTENTS

	Page
1. Victoria and its Roads	... 285
2. One-sized Aggregate	... 286
3. Pre-treatment of Aggregates	... 286
4. Pre-treatment of Binder	... 288
5. Binder Aggregate Relationship	... 289
6. Uniform Spreading of Binder and Aggregate	... 294
7. Pneumatic-tyred Rollers	... 296
8. Conclusions	... 297

1. VICTORIA AND ITS ROADS

1.1. Victoria, the smallest of the States in Australia, is situated in the southern part of the continent. Its area is only about 3 per cent of the whole country. The population of the State is only 2.5 million. Yet, it has 53 lakh of motor vehicles and a road mileage of 110,000, of which 15,000 miles of good metalled roads are classified. Nearly 20 per cent of all the roads and nearly 12 per cent of the classified roads in the country are in the State.

1.2. The classified roads are in charge of the State Country Roads Board and do not include roads in the metropolitan areas. 8,000 miles of the classified roads are black-topped and the programme of the Board comprises treatment or re-treatment of about 1,000 miles every year. To make the best use of the available funds, the Board follows a policy of providing single lane pavement with spray surface dressing. Thus it is able to black top many miles of roads. In the opinion of the Author*, the practice followed in Victoria for surface dressing work has resulted in considerable economy and in providing excellent black-topped surfaces with maximum riding comfort.

*The Author visited Australia in 1953-54 under the United Nations Fellowship Scheme,

1.3. The main features of surface dressing work as practised in Victoria are :

1. use of single-sized aggregate ;
2. pre-treatment of the aggregate ;
3. pre-treatment of the binder ;
4. binder aggregate relationship ;
5. uniform spreading of binder and aggregate ; and
6. use of pneumatic-tyred rollers.

2. ONE-SIZED AGGREGATE

2.1. A bituminous surface dressing is essentially a layer of mineral aggregate put down on a prepared pavement, the aggregate being held in position principally by the adhesive coat of bituminous binder. Surface dressing serves to protect the pavement from wind and water erosion and surface disturbances by wheel loads. At the same time it provides a smooth but nonskid riding surface. It does not add any appreciable strength to the pavement.

2.2. An ideal surface dressing coat will be such that the aggregate particles are all of uniform size, cuboidal in shape and laid shoulder to shoulder with just sufficient bitumen to wet all the faces except the top, like a layer of closely packed sugar cubes. If the aggregate is composed of different sizes of particles, thin and flaky particles rock over each other and are either crushed under traffic or whipped off. The particles at bottom alone which are held in position by the binder are not likely to have the same strength against load and wear and tear as one single particle. So one-sized aggregate is specified for use in surface dressing. An aggregate of which approximately 60 to 70 per cent by weight of the whole material passes a sieve of a specified size and is all retained on a sieve having an opening of seven-tenth of the specified size, is called one-sized aggregate. Specifications for one sized aggregate are given in Appendix I.

3. PRE-TREATMENT OF AGGREGATE

3.1. Adequate adhesion between the pavement and the binder and between the aggregate and the binder is very important. The binder, in its liquid state, wets the pavement and the aggregate and when it cools down to a semi-solid, acts as

the glue. Adhesion of the binder with aggregate depends upon the shape, nature of surface, cleanliness and dryness of the aggregate. Irregular or angular shaped aggregate without sharp edges gives the best results. Again, smooth, glassy, polished or coarsely crystalline surface texture gives poor adhesion. Aggregates, which have a clean and rough sand paper like surface give very good adhesion. If the aggregate is unclean, a layer of dust or other foreign material lies between the aggregate and the binder reducing the adhesion considerably. If the aggregate is wet, the binder does not come into intimate contact with the surface of the stone. So presence of moisture in the aggregate is detrimental to the development of a proper bond between the aggregate and the binder.

3.2. Therefore, aggregates, are given what is called the pre-treatment to improve their adhesion characteristics. The treatment comprises of a light application of oil or oil and an adhesive agent to the aggregate before it is spread on the binder coated pavement. The application of oil to dusty aggregate improves the affinity of stone to the binder and helps to clean the aggregate. If the aggregate is wet, an adhesion agent is mixed in the oil. The adhesion agent improves the bond between binder and the aggregate and assists the binder to come into contact with the surface of stone. Application of adhesion agents to dry clean aggregate, which has a surface of poor adhesion characteristics, increases the strength of the joint between the binder and the aggregate. But pre-treatment of the aggregate does not in any way justify the use of poor, dusty and wet aggregates, whenever these can be avoided.

3.3. The oil used in the pre-treatment of the aggregate is an asphaltic oil, i.e., Diesel Fuel oil, Flux oil or Diesoline. The adhesion agents are chemical compounds called "Cationic detergents". The following have been tried, with a good measure of success.

(a) A commercial mixture of olive and stearic acids. (Oleo-stearic acid) $2\frac{1}{2}$ per cent of the material is incorporated in the bitumen and the aggregate coated with 2 per cent lime.

(b) "No Strip" (1 per cent grade) a commercial adhesion agent of the amonic soap-polyamide. One per cent is added in the bitumen.

(c) "Wet fix" (1 per cent grade) an adhesion agent which is believed to be of the same type as "No Strip".

(d) Cetyl pyridinium bromide, also known Fixanole mixed at 5 per cent in bitumen.

(e) Lead Naphthenate (30-21 per cent grade).

The present practice is to apply adhesion agents by hand to coat the aggregate, in mixture with diesel fuel oil. The procedure adopted in preparing the mixture in the field is to heat the oil to about 200° F. About 10 gallons of hot oil is placed in a 44 gallon drum, to this is added one gallon of adhesion agent which has been warmed and then the balance of the hot oil required is poured into the drum. The drum is agitated by rolling on the ground and the mixture is used as soon as it has cooled.

3.4. The rate of application of the oil or admixture of the oil and the adhesion agent varies with the nature, sizes and type of the aggregate. Generally between one to two gallons per cu. yd. of material is applied. The aim is to ensure that such a quantity of mixture is applied that each stone is uniformly coated, with a film of oil, so that it presents a slightly oil-damp surface, is completely covered, but has no free oil. Excess oil produces overfluxed black fat patches in the finished surface, which have to be avoided. Appendix 2 is a guide to the quantities of oil or the adhesion agent solution to be applied for pre-treating the aggregates.

3.5. In New South Wales half a per cent magnesium stearate mixed in bitumen has been found to considerably improve the affinity of stone to bitumen than to water.

3.6. When an adhesion agent is used, it is essential that the speed of traffic over the finished road is kept low, until water from the wet aggregate has dispersed and adhesion between the binder and the aggregate has been achieved. All traffic is excluded for two hours from the time rolling is complete and in case of rain, for two hours after rain stops.

4. PRE-TREATMENT OF BINDER

4.1. To obtain good adhesion the fluidity of the binder must be low enough to wet the surface of the aggregate and the road under the temperature conditions prevailing at the time of spraying. The wetting power of the binder depends upon its viscosity and surface tension. As the temperature rises the viscosity and surface tension get reduced. Once intimate contact with the binder has been obtained low viscosity gives good wetting and high viscosity good adhesion. Surface tension is independent of viscosity and thickness of film. A low surface tension aids penetration and spreading, and gives high coverage. When hot bitumen is sprayed, it has sufficiently low viscosity and surface tension to wet the aggregate, but it soon cools down to the temperature of the road. So enough

time for full wetting of the aggregate and the road surface is not obtained. Therefore by adding volatile fluxing oils, which more or less evaporate during the setting of the bitumen viscosity and surface tension are kept low temporarily. The fluxing oils are added to the bitumen immediately prior to spraying. The quantity to be added depends mostly on the road temperature and climatic conditions at the time of application. Road temperatures are measured by placing a black bulb thermometer on the road and covering it with sufficient road-side grit to protect it from the wind. Different grades of bitumen have different viscosity at the temperature of their application and the quantity of fluxing oils to be added depends on the nature of the aggregate used and the traffic on the road.

4.2. Two fluxes are added: asphaltic flux oil and power kerosene; one is heavy and the other is relatively volatile. The asphaltic flux oil gives a slower rate of set and the kerosene a relatively rapid rate. The asphaltic flux oil has more or less relatively permanent effect in softening the bitumen, whereas kerosene lowers the viscosity temporarily to give it sufficient time to intimately cover the surface of the road and the aggregate. By adjusting the relative proportions of these two, the vagaries of weaker can be conveniently overcome, as the work proceeds.

4.3. The Country Roads Board has prepared tables to determine the quantity of flux oils for different sizes of aggregates, and traffic and weather conditions. Tables for aggregates of sizes from 1/8 to 3/4 inch are given in Appendix 3.

Oils of the grades suitable for fluxing, particularly kerosene are inflammable, and the heating of the material and its mixing with bitumen need to be carried with precautions against fire. Mixing of kerosene should be done at a place different from that where bitumen is heated. A convenient method is to pump it in the sprayer and circulate. Kerosene should always be added, when the temperature of the mixtures of bitumen and flux oil does not exceed 200°F.

5. BINDER AGGREGATE RELATIONSHIP

5.1. It has been shown by Mr. F. M. Hanson of the New-Zealand Society of Civil Engineers that when the aggregate used is substantially one sized, the amount of bitumen and the quantity of aggregate which should be used in sealing a road surface depend upon the average least dimensions of the aggregate. The

average least dimension is the average of all the least dimensions of the one sized aggregate pieces. This conclusion is based on the assumption that each stone, in a true one sized aggregate, ultimately takes up such a position on the road that it lies with its least dimension vertical.

5.2. The average least dimension* is obtained as follows :

- (a) A representative sample is spread out on a table so that there is no segregation of the various sizes.
- (b) An area containing 100-200 particles is separated from the remainder.
- (c) The least dimension of each of these stones is then measured with callipers. From these measurements the average least dimension is calculated.

Bar screens are available for carrying out the test quickly. The method of using such screens is illustrated by the following example :

The average least dimension is obtained by using a series of screens having rectangular openings of which the smaller dimensions are :

3/4, 1/2, 3/8, 3/16, 1/8 in.

Assume that a sample of 50 oz is tested.

The sample is screened in the ordinary way and then the stones retained on each screen hand picked to ascertain that none can be made to pass through. The stones retained on each screen

*Note 1 :—Mean least dimension is the mean of the clear openings of the two bar screens concerned.

Note 2 :—The method for determining average least dimension applies to stone which is substantially of one size. That is to say stone complying with British Standard Specification No. 63. Where the stone contains an excess of fines it is doubtful whether the method should be applied. All material described as "undersize" in BS. 63 should be omitted from the sample used for the determination of the Average Least Dimension.

are then weighed. The results are recorded as shown in the Table below :

	Weight oz.	Mean least dimension Inch.	Weight divided by mean least dimension.
Passing 3/4 inch retained on 1/2 inch.	20	.625	32.0
„ 1/2 inch retained on 3/8 inch.	12	.437	27.4
„ 3/8 inch retained on 1/4 inch.	9	.312	28.8
„ 1/4 inch retained on 3/16 inch.	5	.219	22.8
„ 3/16 inch retained on 1/8 inch.	4	.156	25.6
Total	50	—	136.6

Average Least Dimension -

$$\frac{\text{Total weight}}{\text{Total of the column weight divided by the mean least dimension}} = \frac{50}{136.6} = 0.37 \text{ in.}$$

5.3. The quantities of binder and aggregate are determined by the F. M. Hanson method as follows :

Assumptions

- (a) The voids in loose layers of one sized aggregates from 1/4 to 1 1/2 in. sizes when spread shoulder to shoulder vary from 46 to 54 per cent of the volume.
- (b) The voids in a loose layer of one sized aggregate can be taken as 50 per cent.
- (c) The approximate average thickness of a loose layer can be determined by dividing the loose volume of aggregate spread by the area of surface covered.

- (d) The void space in a layer of one sized aggregate compacted by a roller is approximately 30 per cent of the compacted depth.
- (e) For all practical purposes the voids in a traffic compacted layer is taken as 20 per cent of the compacted volume. For a traffic compacted $\frac{1}{2}$ in. thick layer the voids may be considered as equal to a depth of $\frac{1}{10}$ in. If a film of bitumen $\frac{1}{10}$ in. thick was first applied the bitumen would ultimately be squeezed up, fill all the voids and rise flush with the average surface, that is to the average height of the traffic compacted layer of aggregate.

5.4. Relationship of Loose Depth to Traffic Compacted Depth

Let the net loose depth be $= N$

Since, this layer contains 50 per cent

voids, voids and solid stone are each $= \frac{N}{2}$

Let the traffic compacted depth be $= X$

Then

(i) Voids in the traffic compacted

depth (depth-solid stone) $= X - \frac{N}{2}$

(ii) Voids in the traffic compacted depth

$= 20 \text{ per cent of } X$

$$\therefore X - \frac{N}{2} = \frac{1}{5} \text{ of } X$$

$$\text{Or } X = \frac{5}{8} \text{ of } N \text{ or } = 0.625 N$$

The traffic compacted depth of a layer of aggregate equals $62\frac{1}{2}$ per cent of the average net loose depth which is equal to the loose volume of the aggregate less the loss due to whip-off divided by the area covered.

5.5. Each piece of aggregate has one dimension which is less than every other dimension. It is the average of these least dimensions which determines the average traffic compacted depth.

Sieve analysis is of little value in determining the coverage value of an aggregate.

With a good quality aggregate although the grading after rolling and traffic compaction is different from that before spreading, the traffic compacted depth still corresponds very closely to the original average least dimension.

Since the traffic compacted depth or average least dimension is $62\frac{1}{2}$ per cent of the net loose depth, the average least dimension of an aggregate multiplied by 1.6 will give the minimum loose depth required without any allowance for whip-off.

5.6. Determination of the Quantity of Binder to be used

A good one sized aggregate will be securely held in position if sufficient binder is provided to fill 50 to 70 per cent of the voids in the traffic compacted work.

One gallon of binder at 60° F. is considered to occupy 276.5 cu. in. or 0.16 of a cu. ft.

- (a) For roads carrying upto 250 vehicles per day filling 69 per cent of the voids in the traffic compacted seal is aimed at.

On the above mentioned basis, for traffic upto 250 vehicles per day the rate of application of binder in gallons per sq. yard equals 0.65 times the average least dimension in inches.

- (b) For roads carrying over 250 vehicles per day the aim is to fill 64 per cent of the voids.

The rate of application of binder in gallons per square yard works out to 0.60 times the average least dimension in inches.

Limitations

The rates of application of binder mentioned above do not include any allowance for the following :

- (i) aggregate which does not conform to the specified grading, is absorbent, is coated or dirty ;
- (ii) a pavement which is hungry, ravelling, absorbent or not properly primed and or constructed with lime stone aggregate ; and
- (iii) work which is a primer seal.

5.7. Determination of the Rate of Application of Aggregate

When using a one sized stone the final cover of aggregate held by the binder will be only one stone thick.

Under normal operating conditions it is not possible to spread the net quantity of aggregate which will finally adhere. The loss of aggregate due to traffic whip-off will vary from 2 to 25 per cent depending mainly upon the type of binder, condition of the aggregate, prevailing weather, spreading equipment and the speed and volume of the traffic. High traffic speed soon after spreading produces heavy whip-off.

With normal working conditions the loss due to traffic whip-off can be taken to be between 8 to 10 per cent. Therefore to obtain the gross loose depth of aggregate to be spread the average least dimension should be multiplied by 1.75 to 1.85. Unless there are any known special conditions, the Country Roads Board uses a factor of 1.75.

Appendix 4. gives the quantities of aggregate and bitumen (80/100) to be applied under different conditions.

6. UNIFORM SPREADING OF BINDER AND AGGREGATE

6.1. For obtaining good results, uniform application of the correct quantity of bitumen and aggregate is very important.

Bitumen is applied by the use of a bitumen pressure spray. The sprayer consists of a tank of 800 gallons capacity, to carry the hot bitumen, mounted on a motor truck chassis, and connected to a spray bar at the rear of the truck. An auxiliary engine to drive a pump, which is fitted with a three-way valve, to allow the hot binder to be pumped into the tank or to be circulated in the tank and the spray bar manifold or to be sprayed on to the road is also mounted on the truck. The speed of the engine can be varied so that the pump maintains a constant pressure. The binder tank is also lagged and fitted with a blow torch to enable the binder to be kept hot at the desired temperature. The rate of spraying depends upon the speed of the truck, which is accurately measured by a special tachometer mounted in the cabin of the driver. The meter is operated by a light wheel, which can be lowered to make contact with the road, when required. The spray bar is adjustable so that varying widths of road can be sprayed in a single run, with constant pressure on the spray bar and proper regulation of the speed of sprayer, the bitumen can be sprayed within 5 per cent of a specified rate of application.

6.2. The surface is first cleared with a mechanical broom and the sprayer, aggregate spreading trucks and the roller are kept ready in position. Spraying is then commenced from one

end of the road in full width, in a measured length depending upon the quantities of materials for one operation. This is followed by the spreading of aggregate, the trucks travelling in reverse direction followed by the roller. When taking up the next length of spray, the previously sprayed and coated surface is covered with paper stretched across the road to have a clean joint without any overlapping. The whole process is amazingly quick and efficient. Depending upon the lead of binder, which comes directly from the refineries, on an average 1,50,000 sq. ft. of new surface can be conveniently treated in a day by about a dozen men.

6.3. A complete record is kept of each spray including its exact location in the pavement, the amount of binder, flux oils and aggregates used, the road temperature and weather conditions. For this purpose, a spraying field book is kept at the site of the work, in which entries are made as the work proceeds, and on the completion of the job it is returned to the concerned officer for record.

6.4. The aggregate is spread on the bitumen sprayed surface by a special aggregate sprayer developed by the Country Roads Board, Victoria. It is a box on wheels with a rubber belt stretched between rollers set in the bottom of the box. The belt is driven from the road wheels and has an adjustable cut off gate above the belt. The farthest edge of the belt is beyond the angle of spill when the cut off plate is at its maximum opening.

The truck bringing aggregate butts against the unit, pushing it and discharging its load into the hopper. By adjustment of the cut off plate opening and speed of the truck, the desired quantity of aggregate can be spread per unit surface area. Each spreader is capable of spreading, say a 3/4 in. grit at 5 cu. ft. per 100 sq. ft. over an area of 30,000 sq. ft. in one hour. A distribution to within 5 per cent from the required mean can be attained. A sketch of the unit is shown in Fig. 1.

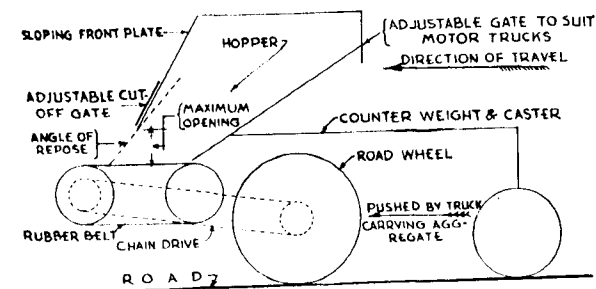


Fig. 1. Rotating belt spreader

6.5. In the State of New South Wales, spreading of the aggregate is carried out by tipping trucks of approximately 135 cu. ft. capacity, fitted with a special tail gate and fan tail spreaders. The rate of spreading is dependent upon the opening of the tail gate and the speed of the truck. The under or over-spread of material does not exceed by five per cent when experienced drivers and tail gate operators are employed. In practice a slight under spreading is aimed at so that any areas of thin

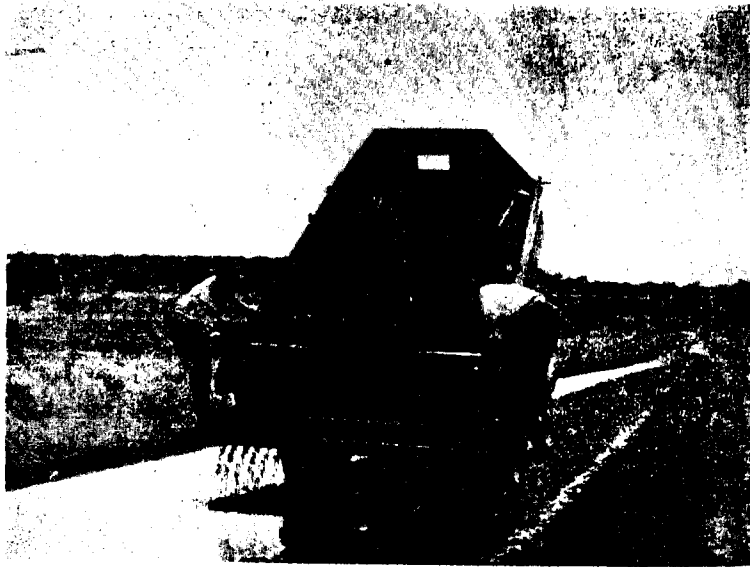


Photo 1. Spreading aggregate through truck tail gate & fan tail spreader

spread can be made up by hand. This arrangement is comparatively much simpler than the one used in Victoria and can be usefully adopted in India. Photo 1 shows the fan tail spreader.

7. PNEUMATIC-TYRED ROLLERS

7.1. Pneumatic-tyred rollers originally came into use for consolidation of gravel or stabilized soil. For bituminous surfacing work they are not being used extensively as yet, though their use is strongly advocated. The main advantage in using pneumatic-tyred rollers is that they are capable of consolidating

the surface slowly and uniformly, without causing damage to the aggregate or the pavement in general. For rolling bituminous surface dressing their use is recommended for two reasons. Firstly, they are able to knead the aggregate in the binder, without breaking or crushing the aggregate pieces, which is very important. Secondly, as the strength of the aggregate binder joint is usually weak for sometime after spraying, slow moving pneumatic tyres do not have the tendency to dislodge the aggregate. On steep grades, or short radius curves, they have been found particularly useful. The function of a roller in bituminous surface dressing is only to press the aggregate into position in the binder without causing damage and not to provide mechanical interlock. Light steel-tyred rollers, it is believed should also be able to have an equally satisfactory performance as the pneumatic-tyred rollers.

7.2. A pneumatic-tyred roller consists of a small truck with two or more axles with a number of pneumatic-tyred wheels. The truck can be loaded to provide the intensity of pressure that is desired.

8. CONCLUSIONS

8.1. In the opinion of the Author the quality and durability of surface dressing work can be considerably improved if the methods described are adopted in India. Also, it would be possible to reduce the quantity of bitumen used and thereby reduce costs. Uniformity of application is very important specially when quantity of material applied is the minimum required. Use of mechanical means for spraying bitumen and spreading of aggregate will not only increase the out-turn but ensure uniformity of application.

8.2. In India surface dressing work is generally limited to the hot months of the year. If mechanical means are adopted it will be possible to do many more miles during the hot weather resulting in considerable reduction in the quantity of fluxing oils to be used and full utilisation of rollers. The Author had an opportunity of inspecting many surface dressed roads in Victoria during and after the execution of the work. The finished surface was extremely even and presented a coarse sand-paper appearance. Re-painting of such surfaces is generally done after 5 to 7 years in Victoria in spite of the traffic being very heavy and they have been found to have behaved very well and provided excellent riding surfaces.

8.3. In India there are considerable number of iron-tyred bullock carts on the road, whereas the traffic in Australia is essentially pneumatic-tyred. However, with good quality of aggregate and good workmanship, in the opinion of the Author, the presence of the iron-tyred traffic may not have much adverse effect on surface dressing.

Appendix 1.

Specifications for one sized Aggregate

General Description			Specification											
Nominal size	Nominal range of sizes (sizing)		Tolerance on oversize				Specified size Tolerance on nominal range			Tolerance on undersize			Shape Flakiness Index	
	All passing a sieve of	All retained on a sieve of	Material Passing		Material Retained on		Passing a sieve of	Retained on a sieve of	Per cent by weight minimum or range	Material Passing		Per cent by weight minimum		
			a sieve of	Per cent by weight	a sieve of	Per cent by weight maximum				a sieve of	Per cent by weight maximum		a sieve of	Per cent by weight maximum
2½ in.	2 in.	2 in.	3 in.	100	2½ in.	15	2½ in.	2 in.	65	1½ in.	5	7	0.5	40
2 in.	1½ in.	1½ in.	2½ in.	100	2 in.	15	2 in.	1½ in.	70	1 in.	5	7	0.5	40
1½ in.	1 in.	1 in.	2 in.	100	1½ in.	15	1½ in.	1 in.	75	¾ in.	5	7	0.5	40
1 in.	¾ in.	¾ in.	1½ in.	100	1 in.	15	1 in.	¾ in.	55	¾ in.	7	7	0.5	35
¾ in.	½ in.	½ in.	1 in.	100	¾ in.	15	¾ in.	½ in.	65	¾ in.	7	7	0.5	35
¾ in.	½ in.	½ in.	1 in.	100	¾ in.	5	¾ in.	—	35-65	¾ in.	15	7	0.5	35
¾ in.	½ in.	½ in.	¾ in.	100	¾ in.	15	¾ in.	¾ in.	55	¾ in.	7	7	0.5	35
¾ in.	½ in.	½ in.	¾ in.	100	¾ in.	15	¾ in.	¾ in.	60	¾ in.	10	7	1	35
¾ in.	¾ in.	No. 10	¾ in.	100	¾ in.	15	¾ in.	¾ in.	70	7 mesh	10	25	2	—
¾ in.	¾ in.	No. 10	¾ in.	100	¾ in.	15	¾ in.	10 mesh	65	14 mesh	10	25	2	—
Air blown grit	No. 7	No. 14	¾ in.	100	7 mesh	15	7 mesh	14 mesh	70	25 mesh	2	100	0.5	—

All sizes and sizings are according to British Standard.

Abbreviations: e.g., No. 7 or 7 mesh refers to a British Standard square mesh sieve of nominal mesh No. 7.

Appendix 3

Quantities of Fluxes for Different Aggregates and Traffic and Weather Conditions

(For 100 parts of 80/100 Penetration Bitumen)

Range of air temper- ature in deg. F.	Binder Viscosity Stocks at 122° F. approximate	Fluxes for 100 parts of Bitumen by volume @ 60° F. for traffic in vehicles per day					
		Below 250		250 to 1,500		Over 1,500	
		Asphal- tic oil	Power kero- sene	Asphal- tic oil	Power kero- sene	Asphal- tic oil	Power kero- sene
1/8, 3/16 and 1/4 in. Aggregates							
40—50	10	7½	25	5	25	2½	30
50—60	20	7½	17½	5	20	2½	22½
60—70	40	7½	12½	5	15	2½	17½
70—80	75	7½	10	5	12½	2½	15
80—90	150	7½	7½	5	7½	2½	12½
Over 90	350	7½	2½	5	5	2½	7½
3/8 and 1/2 in. Aggregates							
40—50	15	7½	20	5	22½	2½	25
50—60	30	7½	15	5	17½	2½	22½
60—70	65	7½	10	5	12½	2½	15
70—80	150	7½	7½	5	7½	2½	12½
80—90	350	7½	2½	5	5	2½	7½
Over 90	750	7½	—	5	2½	2½	5
5/8 and 3/4 in. Aggregates							
40—50	22½	7½	17½	5	15	2½	22½
50—60	50	7½	15	5	15	2½	17½
60—70	150	7½	7½	5	10	2½	12½
70—80	350	7½	2½	5	5	2½	7½
80—90	750	7½	—	5	—	2½	5
Over 90	1250	7½	—	5	—	2½	2½

Rate of Application of Adhesion Agent Solution or Oil only when no Adhesion Agent is Required
(Gallons per 100 cu. ft. of Aggregate)

Size of the Aggregate	Condition of the Aggregate before treatment					
	Dirty			Clean		
	Dry	Wet	Wet	Dry	Wet	Wet
	Porous	Rough and hard	Smooth and hard	Porous	Rough and hard	Smooth and hard
¾, ¾, or ½ in.	7 to 8	6	5	6	5	4 to 5
¾ or ½ in.	8	7	6	6	5	4 to 5
Coarse sand	—	—	—	—	—	6 to 8
Fine sand	—	—	—	—	—	7 to 9

* Note :—When clean smooth hard wet stone is used, and the application of mixture is 3 gallons per 100 cu. ft., the quantity of adhesion agent used per drum of oil shall be twice the normal quantity.

Appendix 4

Rates of Application of Binder and Aggregate*

Surface Painting, prime and seal coat—single application	For traffic — vehicles per day					
	Below 500		Above 500			
	For traffic — vehicle per day					
Repainting, resealing—single application	Below 250		250 to 1,000		Above 1,000	
	70		65		60	
Percentage of voids to be filled with binder	0.66		0.61		0.56	
Factor by which the average least dimension (inches) is multiplied						
Binder lb per 100 sq. ft.	Average least dimension, Inches	Aggregate sq. yd. per 1 cu. yard (loose)	Average least dimension, Inches	Aggregate sq. yd. per 1 cu. yard (loose)	Average least dimension, Inches	Aggregate sq. yd. per 1 cu. yard (loose)
11.55	0.15	140	0.15	140	0.15	140
	0.16	130	0.16	130	0.16	130
	0.17	120	0.17	120	0.17	120
14.44	0.18	115	0.18	115	0.18	115
	0.19	110	0.19	110	0.19	110
	0.20	105	0.20	105	0.20	105
17.33	0.21	100	0.21	100	0.21	100
	0.22	95	0.22	95	0.22	95
	0.23	90	0.23	90	0.23	90
	0.24	85	0.24	85	0.24	85
	0.25	80	0.25	80	0.25	80
20.22	0.26	75	0.26	75	0.26	75
	0.27	70	0.27	70	0.27	70
	0.28	65	0.28	65	0.28	65
	0.29	60	0.29	60	0.29	60
	0.30	55	0.30	55	0.30	55
23.11	0.31	50	0.31	50	0.31	50
	0.32	45	0.32	45	0.32	45
	0.33	40	0.33	40	0.33	40
	0.34	35	0.34	35	0.34	35
	0.35	30	0.35	30	0.35	30
26.00	0.36	25	0.36	25	0.36	25
	0.37	20	0.37	20	0.37	20
	0.38	15	0.38	15	0.38	15
	0.39	10	0.39	10	0.39	10
	0.40	5	0.40	5	0.40	5
28.44	0.41	0	0.41	0	0.41	0
	0.42	0	0.42	0	0.42	0
	0.43	0	0.43	0	0.43	0
	0.44	0	0.44	0	0.44	0
	0.45	0	0.45	0	0.45	0
31.77	0.46	0	0.46	0	0.46	0
	0.47	0	0.47	0	0.47	0
	0.48	0	0.48	0	0.48	0
	0.49	0	0.49	0	0.49	0
	0.50	0	0.50	0	0.50	0
34.33	0.51	0	0.51	0	0.51	0
	0.52	0	0.52	0	0.52	0
	0.53	0	0.53	0	0.53	0
	0.54	0	0.54	0	0.54	0
	0.55	0	0.55	0	0.55	0
37.55	0.56	0	0.56	0	0.56	0
	0.57	0	0.57	0	0.57	0
	0.58	0	0.58	0	0.58	0
	0.59	0	0.59	0	0.59	0
	0.60	0	0.60	0	0.60	0
40.44	0.61	0	0.61	0	0.61	0
	0.62	0	0.62	0	0.62	0
	0.63	0	0.63	0	0.63	0
	0.64	0	0.64	0	0.64	0
	0.65	0	0.65	0	0.65	0

* See notes on page 304.

Appendix 4 (Contd.)

NOTES

- (a) The rates of application of binder apply to all mineral aggregates other than that produced from limestone.
- (b) The rates are inclusive of allowance for whip-off found necessary in practice, that is, the rates at which the binder and aggregate shall be applied.

When purchasing aggregate, obtain an additional 7½ per cent for wastage, etc.

Surface Painting - Prime and Seal

Aggregate	Pavement Material	
	Other than Limestone	Limestone
Other than Limestone	As per Table	Add 2.89 lb per 100 sq. ft. to rate given in the Table.
Limestone	Add 2.89 lb per 100 sq. ft. to rates given in the Table	Add 5.78 lb per 100 sq. ft. to rates given in the Table.

Repainting - Re-seal

Aggregate	Surface Condition of Pavement			
	Hungry	Smooth	Black (1)*	Black (2)†
Other Than Limestone	Add 2.89 lb per 100 sq. ft. to rates given in the Table	As per rates given in the Table		Deduct 2.89 lb per 100 sq. ft. to rates given in the Table
Limestone	Add 5.78 lb per 100 sq. ft. to rates given in the Table	Add 2.89 lb per 100 sq. ft. to rates given in the Table		As per notes given in the Table

* (1) When the application from the Table is less than 31.77 gallon per sq. yd.

† (2) When the application from the Table is 31.77 gallon per sq. yd. or greater.

RESEARCH SECTION

LABORATORY INVESTIGATION ON THE EFFECT OF ADDITION OF SOME SURFACE ACTIVE AGENTS TO STABILIZED SOIL MIXTURES

By

I. S. UPPAL & O. P. WASON

[Following precedent, the Note received for publication was circulated to the Members of the Research Organization Division for comments. The comments received were sent to the Authors for their reply. The Note along with the comments and the reply of the Authors were considered at the meeting of the Research Organization Division held at Delhi on 12-12-56. In accordance with the decision of the Division the Note is now published for information of Members.]

1. INTRODUCTION

1.1. High resistance to the softening action of water, compressive strength and low volumetric shrinkage are desirable characteristics of a compacted and stabilized soil. As the degree of compaction increases, there is improvement in the above mentioned properties. The optimum moisture content, necessary for producing the maximum dry density, varies with the nature of the soil. Also, in order to reduce to the minimum the shrinkage, on drying, of the stabilized mass without adversely affecting its density and strength, it is desirable to keep the optimum moisture as low as possible.

1.2. The addition of a suitable surface active agent to water may be expected to facilitate the wetting of soil particles resulting in lower optimum moisture without affecting density and other properties or may yield increased density with the optimum moisture. The presence of surface active agent in water renders the liquid less coherent within itself and more adherent to the solid surface, thus promoting wetting of the solid by causing the liquid to spread upon the solid surface and to penetrate minute pores. Improvement in the wetting of the surfaces of the soil particles can be expected to result in improvement in the properties of stabilized soil.

Therefore, investigation on the effect of four commonly used surface active agents on the engineering properties of different soils have been carried out. These were:

- (1) Soap
- (2) Sulphated castor oil (turkey red oil)
- (3) Soapnuts, and
- (4) Wetting No. 1.

2. THE SURFACE ACTIVE AGENTS STUDIED

2.1. Soap

Is the salt of long chain fatty acids derived from naturally occurring fats and oils. It possesses the disadvantage of being unstable to dilute acids and soluble calcium and magnesium salts. 'Sunlight soap' was selected for the purpose of this study. The soap was dried in the oven to constant weight and a stock solution prepared for use in experimental work.

2.2. Turkey Red oil or Sulphated Castor Oil.

This is a viscous oily transparent liquid obtained by the action of concentrated sulphuric acid on castor oil. Its chemical formula is $CH_3(CH_2)_5\overset{\textstyle |}{\underset{\textstyle O-SO_3H}{CH}}CH_2CH=CH(CH_2)_7COOH$.

This is more stable to dilute acids and hard waters than the soap. It is capable of forming relatively insoluble calcium and magnesium salts and is extensively used as a wetting and softening agent in textile industry and as a fat lacquering agent in leather industry. The turkey red oil used in this study was prepared in the laboratory according to Uppal's method^{3,7} of sulfation at 50–55 deg. C.

The resultant product was found to contain:

Fatty matter	Per cent	85
Combined SO_3	" "	5.2

2.3. Soap-Nut Powder

Soap-nut, commonly known as *Ritha*, is the fruit of a tree found in hilly areas almost throughout India upto an altitude of 5,000 feet. Kangra district in the Punjab alone is reported to be producing over 3,000 tons of soap-nuts annually. The fruit consists of two parts, (a) pericarp which forms 57 per cent and (b) black seed

forming the remaining 43 per cent. Saponine ($C_{26}H_{48}O_{10}$) are the main constituents of pericarp which was used in this study. The present use of soap-nut in this country is limited to household washing of woollen goods and to washing of gold and silver ornaments. Due to its good wetting properties and great stability to hard waters, its industrial utilization as a textile auxiliary has been recommended by Uppal and Mehta⁴. The pericarp was dried and ground to pass 10 mesh and used.

2.4. Wetting Agent No. 1

This is an ethylene oxide condensation product in liquid form and is very stable to acids and metallic salts. Due to its high wetting power and unreactivity with the materials usually used in constructional work, it is finding extensive use in the cement concrete mixes to improve their properties, e.g., the increased fluidity, workability, and strength.

2.5. The surface active agents were first dissolved in the requisite quantity of water (optimum moisture) which was then mixed well with soils. The method of compaction was that adopted by Mehra and Uppal⁶.

3. SOIL USED IN THE STUDY

The effect of increasing quantities of all the four surface active agents mentioned above was studied on four types of soils commonly occurring in the Punjab. The chemical and physical characteristics of these soils are given in Table 1. The particle

TABLE 1

Physical and Chemical Characteristics of the Soils used

Sl. No.	Description	Liquid limit	Plasticity index	Sand content per cent	Optimum moisture per cent	Sulphate per cent	Total solids	Exchangeable		pH value
								Ca, m.e. 100 gm	Na+k gm m.e. per 100 gm	
1.	Sandy loam	23.5	6.2	66.5	12	Nil	0.55	7.5	3.44	7.2
2.	Silty loam	33.6	11.7	9.1	14	Nil	0.045	10.5	3.86	7.2
3.	Silty-clay loam	36.4	16.1	8.8	12	Nil	0.045	14.5	3.02	7.4
4.	Loam	27.1	9.0	47.2	12	Nil	0.045	9.5	2.60	7.5

size distribution is shown in Fig. 1.

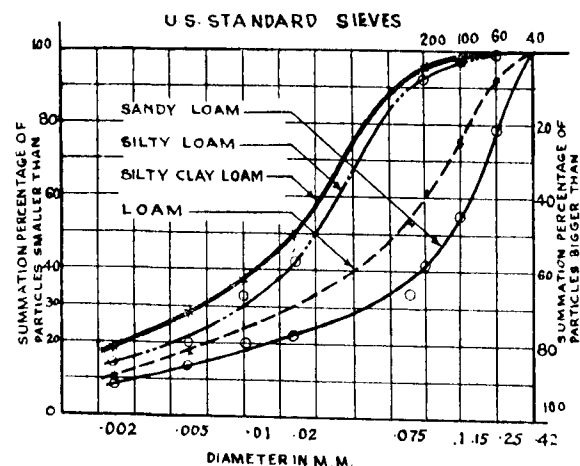


Fig. 1. Particle size distribution of soils tested

4. EFFECT OF ADDITION OF SURFACE ACTIVE AGENTS ON OPTIMUM MOISTURE AND DENSITY

Optimum moisture for each soil was first determined by compacting 200 gm of the soil into blocks in the compaction apparatus. The process was repeated with increasing quantities of the different surface active agents in water to see whether there was any change in optimum moisture and density due to their addition. The results are shown in Table 2 and graphically represented in Fig. 2.

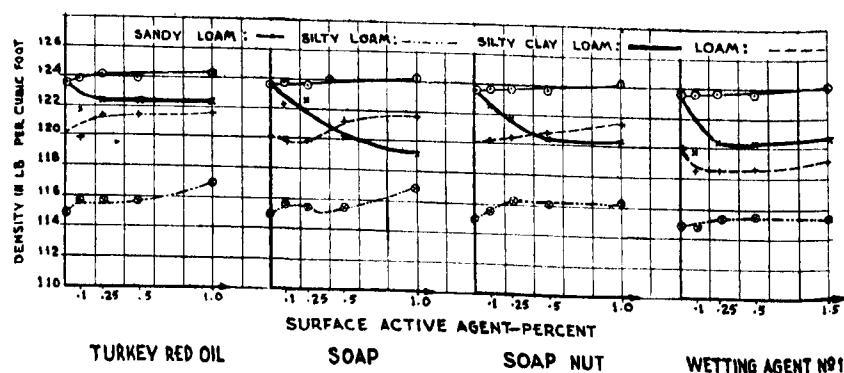


Fig. 2. Effect of surface active agents on the density of compacted soils

TABLE 2
Optimum Moisture and Density of different Soils with varying Percentages of Surface Active Agents

Surface active agent Percentage	Sandy Loam		Silty Loam		Silty-Clay Loam		Loam	
	Optimum Moisture	Density lb per cu. ft.	Optimum Moisture	Density lb per cu. ft.	Optimum Moisture	Density lb per cu. ft.	Optimum Moisture	Density lb per cu. ft.
Turkey Red Oil	0	123.55	14	114.80	12	123.55	12	120.12
	.10	123.55	14	115.43	12	121.68	12	119.81
	.25	123.86	13	115.43	12	122.30	12	121.37
	.50	123.67	13	115.74	11	122.30	12	121.37
Soap Solution	1.0	124.18	12	116.99	11	122.30	12	121.68
	0	123.55	14	114.8	12	123.55	12	120.12
	.10	123.67	14	115.43	12	122.30	12	119.81
	.25	123.55	14	115.43	12	122.61	12	119.81
Soap-Nut	.50	123.99	14	115.43	12	120.43	12	121.06
	1.0	124.18	14	116.80	12	119.18	12	121.68
	0	123.55	14	114.80	12	123.55	12	120.12
	.10	123.55	14	115.43	12	122.30	12	120.31
Wetting Agent No. 1	.25	123.67	14	116.18	12	121.68	12	120.43
	.50	123.55	14	116.05	12	120.43	12	120.43
	1.0	124.18	14	116.05	12	120.43	12	121.62
	0	123.55	14	114.80	12	123.55	12	120.12
	.10	123.55	14	114.80	12	119.80	12	118.56
	.25	123.55	14	115.43	12	120.30	12	118.56
	.50	123.55	14	115.43	12	120.43	12	118.66
	1.0	123.93	14	115.43	12	120.61	12	119.18

It would be seen that the addition of surface active agents increased the density of the compacted mass except in silty clay loam. The increase in density due to the addition of wetting agent No. 1 was, however, negligible. The increase in density was more pronounced in graded and silty soils than in the sandy soil. Amongst the surface active agents tried, turkey red oil gave the best results, next in order of merit being soap, soap-nut powder and wetting agent No. 1 successively.

The density of silty-clay loam, however, decreased with the addition of increasing percentages of the wetting agents. Of the four wetting agents tried, the decrease in density when turkey red oil was added was least. Also, the addition of turkey red oil resulted in a decrease in the optimum moisture.

5. EFF OF ADDITION OF SURFACE ACTIVE AGENTS ON VOLUMETRIC SHRINKAGE

5.1. The volumetric shrinkage on drying of the blocks prepared in the series of experiments described in para. 4 was also determined. The reduction in volumetric shrinkage due to the addition of surface active agents to different soils is given in Table 3 and Fig. 3. It would be seen that there was significant improvement in the shrinkage of the sandy and graded soil blocks

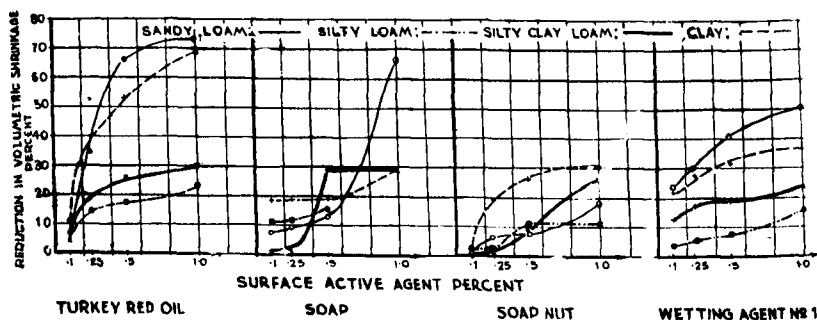


Fig. 3. Effect of surface active agents on the volumetric shrinkage

when turkey red oil was added. Presence of even 0.25 per cent of this surface active agent reduced the shrinkage to about 50 per cent in loamy soils. Soap was the second best next in order of merit being wetting agent No. 1 and soap-nut powder. The effect of surface active agents on reduction of volume shrinkage was least in silty soil, though there was a definite improvement.

TABLE 3

Volumetric Shrinkage of different Soils with varying Percentages of Surface Active Agents

Surface active agent percentage	Sandy Loam		Silty Loam		Silty-Clay Loam		Loam	
	Vol: Shrinkage	Percentage reduction in Vol: Shrinkage	Vol: Shrinkage	Percentage reduction in Vol: Shrinkage	Vol: Shrinkage	Percentage reduction in Vol: Shrinkage	Vol: Shrinkage	Percentage reduction in Vol: Shrinkage
Turkey Red Oil	0.00	1.831	0.0	2.25	2.83	0.0	1.84	0.0
	.10	1.71	6.6	2.1	2.7	4.6	1.66	9.8
	.25	1.10	34.4	1.92	2.3	18.7	.87	52.7
	.50	0.608	66.7	1.865	2.1	26.0	.87	52.7
	1.00	0.51	72.1	1.73	2.0	29.3	.58	68.4
Soap Solution	0.00	1.831	0.0	2.25	2.8	0.0	1.84	0.0
	.10	1.7	7.1	2.0	2.8	1.1	1.5	18.5
	.25	1.66	9.3	1.99	2.8	1.1	1.5	18.5
	.50	1.60	12.6	1.91	2.01	29.0	1.5	18.5
	1.00	0.71	66.7	2.11	2.0	29.3	1.3	29.3
Soap-Nut	0.00	1.831	0.0	2.25	2.83	0.0	1.84	0.0
	.10	1.80	1.7	2.2	2.82	.4	1.8	2.1
	.25	1.71	6.6	2.2	2.8	1.1	1.49	19.0
	.50	1.70	7.1	2.0	2.6	8.1	1.36	26.1
	1.00	1.50	18.0	2.0	2.1	26.0	1.28	30.4
Wetting Agent No. 1	0.00	1.831	0.0	2.25	2.83	0.0	1.84	0.0
	.10	1.40	23.5	2.17	2.48	12.4	1.44	21.7
	.25	1.29	29.5	2.13	2.35	17.0	1.33	27.7
	.50	1.10	40.0	2.09	2.3	18.7	1.25	32.0
	1.00	0.91	50.0	1.90	2.2	22.3	1.17	36.14

6. EFFECT OF ADDITION OF SURFACE ACTIVE AGENTS ON RESISTANCE TO THE SOFTENING ACTION OF WATER

6.1. Blocks made with all the four soils using varying quantities of different surface active agents were dried in shade for a week and then in an oven to constant weight. They were immersed in water at a temperature of 25 deg. C. and the time of disintegration was noted. Blocks which retained their form for five hours were taken out after this period, dried and the percentage loss in weight determined. The results are given in Table 4.

The addition of turkey red oil and soap considerably improved the resistance of a soil to the softening action of water. The sandy soil responded best to this treatment. The effect on graded soil was also quite marked. Generally, Turkey Red Oil was again found to be better than soap. In clayey and silty-clay loams, however, soap behaved better than turkey red oil. Improvemnt with soap nut powder was not very appreciable, while the addition of wetting agent No. 1 appeared to help the disintegration of the blocks.

7. EFFECT OF SURFACE ACTIVE AGENTS ON THE COMPRESSIVE STRENGTH OF STABILIZED SOIL

7.1. Blocks of the four soils using increasing quantities of the four surface active agents were prepared using the modified Abbot's cylinder and tested for compressive strength in the Carver's Laboratory Press. The results are shown in Table 5 and Fig. 4.

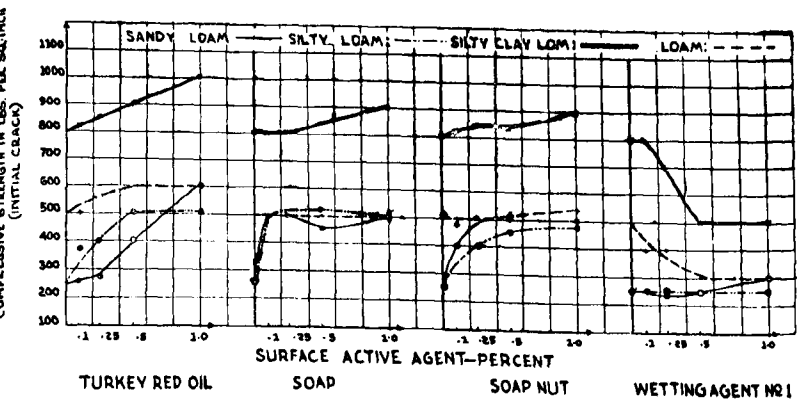


Fig. 4. Effect of surface active agents on the compressive strength of soils

TABLE 4
Resistance of Soils with varying Percentages of Surface Active Agents to the Softening Action of Water

Surface active agent per cent	Sandy Loam		Silty Loam		Silty-clay loam		Loam	
	Period of disintegration minutes	Loss of weight after 5 hours per cent	Period of disintegration minutes	Loss of weight after 5 hours per cent	Period of disintegration minutes	Loss of weight after 5 hours per cent	Period of disintegration minutes	Loss of weight after 5 hours per cent
Turkey Red Oil	38		29		130		22	
	41	2.2	47		131		25	8.2
	—	0.75	75	20.3 Most of the blocks disintegrated	210	6.0 The blocks cracked badly	—	7.3
	—	0.75	—		250		—	5.3
Soap Solution	—		—		—		—	
	38		29		130		22	
	91		90		226		38	
	253	4.30	210	2.5	—	11.5	66	23.2
Soap-Nut	—	0.87	280		—	4.65	—	3.85
	—		—		—	4.5	—	
	38		29		130		22	
	32		46		88		17	
Wetting Agent No. 1	34		50		82		19	
	51		65		230		27	
	11		86		230		34	
	38		29		130		22	
	27		31		90		17	
	28		30		90		20	
	28		35		95		20	
	29		38		98		28	

TABLE 5

Compressive Strength of different Soils with varying Percentages of Surface Active Agents

Surface active agent	Percentage	Sandy Loam lb per sq. in.		Silty Loam lb per sq. in.		Silty-Clay Loam lb per sq. in.		Loam lb per sq. in.	
		Initial	Final	Initial	Final	Initial	Final	Initial	Final
Turkey Red Oil	—	250	695	250	660	800	1425	500	560
	0.10	250	700	372	744	820	1500	500	660
	0.25	270	750	400	750	850	1640	560	745
	0.50	400	800	500	775	900	1800	600	845
	1.00	600	1100	500	1000	1000	1900	600	1050
Soap Solution	0	250	695	250	660	800	1425	500	560
	0.10	500	744	500	770	800	1400	500	740
	0.25	500	800	500	780	800	1450	500	770
	0.50	450	820	520	800	850	1600	500	750
	1.00	500	968	500	800	900	1650	500	750
Soap-Nut	0	250	695	250	660	800	1425	500	560
	0.10	400	800	400	748	820	1400	480	560
	0.25	500	850	400	800	850	1450	500	600
	0.50	500	930	450	800	850	1500	520	600
	1.00	500	1000	480	820	900	1600	550	620
Wetting Agent No. 1	0	250	695	250	660	800	1425	500	560
	0.10	250	500	244	744	800	1200	400	660
	0.25	240	480	250	560	700	1220	400	660
	0.50	250	500	250	420	500	970	300	520
	1.00	300	500	250	300	500	750	300	520

The addition of Turkey Red oil and soap substantially improved the compressive strength of all the four soils. Turkey Red oil was again superior to soap. Soap was, however, more effective than Turkey Red oil when added in small quantities. The beneficial effect of soap-nut extract was not marked while wetting agent No. 1 tended to decrease the compressive strength.

CONCLUSIONS

The addition of small quantities of anionic surface active agents like soap and sulphonated castor oil to the water to bring it to optimum moisture tends to increase the dry bulk density, resistance to the softening action of water, and the compressive strength and reduce volumetric shrinkage on drying of the compacted soil. So, they can be usefully employed as 'aids' to soil stabilization. These are easily available at low

cost and are simple to manufacture. Therefore, they can be prepared at the site of work. Their cost generally varies between Rs 0-8-0 and 0-12-0 per lb.

It has been observed^{5,6} that the addition of cement or lime for the stabilisation of soil has a tendency to reduce the dry bulk density of the resultant mass. Addition of small quantity of the surface active agent in such mixtures can be expected to minimise the decrease in density and thus give more strength to the compacted mass.

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COMMENTS

Shri K. F. Antia suggested that the liquid and plastic limits of the soils to which the surface active agents had been added might be determined to give an indication of the improvement in the capillary and the cohesion characteristics.

Soap solution was perhaps the most satisfactory agent. It produced the largest resistance to the action of water and was the cheapest of all the materials tested.

Although the density of the soils did not increase substantially by the addition of any of the four agents tested (in silty-clay loam, there was a decrease), there was improvement in other desirable properties which would justify the use of the agents, particularly if the cost was negligible as in the case of soap solution. He suggested that large scale trials should be made.

Shri S. L. Kumar remarked that the reduction in the optimum moisture content was not very significant for soils normally utilized for stabilization purposes, as the optimum moisture content for such soils was comparatively low. The optimum moisture content was large for highly clayey soils and soils with peculiar clay minerals such as Montmorillonite, Nontorite, etc., but such soils were rarely used for stabilization.

The increase in compressive strength was also not very appreciable. In fact, the variations might even be considered as experimental deviations.

Reduction in shrinkage was not important, except for very high clayey soils or soils with certain peculiar clay minerals. In view of the fact that soil stabilization was generally restricted to soils with not much of clayey fractions, the importance of reducing shrinkage was not very evident. It was well known that use of soap solutions etc. reduced shrinkage as they decreased the surface tension of water in soils high in silica.

Work, which was not completed as yet, at the Railway Research Station at Lonavla, on the physico-chemical conception of optimum moisture content showed that for stabilization purposes, soils having optimum moisture contents higher than about 12 per cent should be generally avoided. Also, characteris-

tics such as good grading, low plasticity index and a small percentage of clay were desirable. It was important that stabilized soil should satisfactorily stand erosion as well as rise of moisture by capillarity etc., to serve either as building material or as a base course for roads. It was found that use of emulsions, which were likely to be washed away by water, do not give any stability to soils when compacted and exposed for some time. It was obvious that soap solutions, etc., would get washed away and leave the soils in much the same vulnerable condition to erosion and capillary rise. Resins, asphaltic material, or cements were satisfactory, even though cement even did not prevent capillary rise well. It would appear that employing soap solutions etc. which were likely to be washed away by water, would not be of much practical value.

AUTHORS' REPLY

As suggested by **Shri K. F. Antia**, the liquid and plastic limits would be determined and the results reported in the second part of the work which was in progress. It was expected that field trials would be undertaken shortly.

Referring to the comment of **Shri S. L. Kumar**, about the reduction in the optimum moisture content, the Authors stated that the moisture content played a very important part in the density and strength of the stabilized soil. It was always desirable to keep the optimum moisture as low as possible to minimise shrinkage on drying. The addition of some surface active agents proved beneficial in this respect. Table 2, page 311 showed that the optimum moisture of silty soil with no surface active agent was 14 per cent, and at that moisture the density obtained was 1.84. But when one per cent of Turkey Red oil was added, the optimum moisture was reduced to 12 per cent i.e. 14 per cent less and density increased from 1.84 to 1.87.

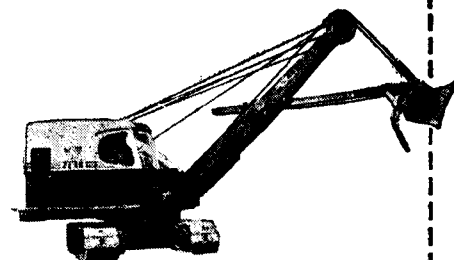
Generally, walls made of stabilized soil developed cracks. This defect was primarily attributable to shrinkage. The addition of a small quantity of Turkey Red oil could be expected to eliminate such cracks. The investigation had shown that Turkey Red oil had also shrinkage reducing properties.

The results given in Table 4 showed that the addition of Turkey Red oil and soap considerably increased the resistance of soils to the softening action of water. Soap and Turkey Red oil

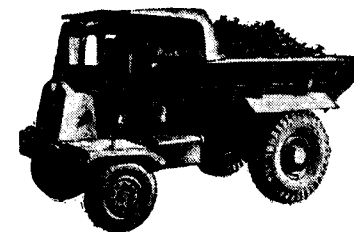
being anionic surface active agents formed some insoluble compounds in the soil mixture and increased its resistance to the softening action of water. So, the apprehension of Shri Kumar that the soap solution etc. would wash away and leave the soil in much the same vulnerable condition did not appear to be correct.

The results presented in the Note showed that in some cases the addition of Turkey Red oil to sandy and silty loams could render them fit for use without the addition of clayey soil.

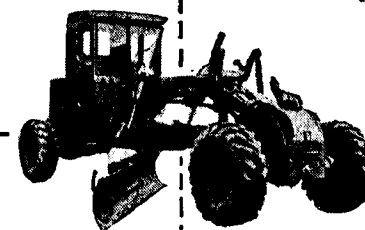
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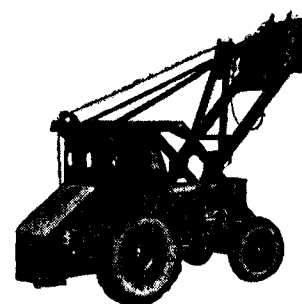
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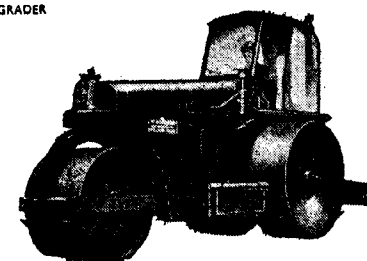
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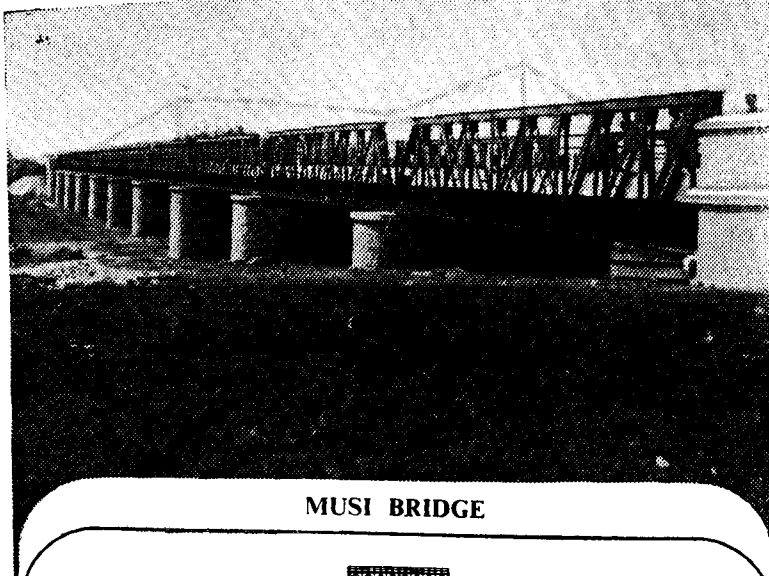
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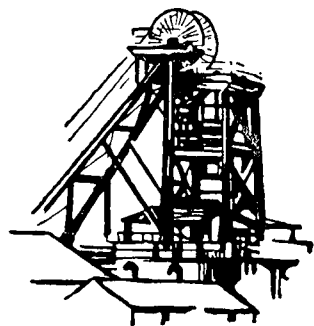
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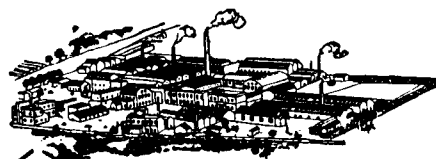
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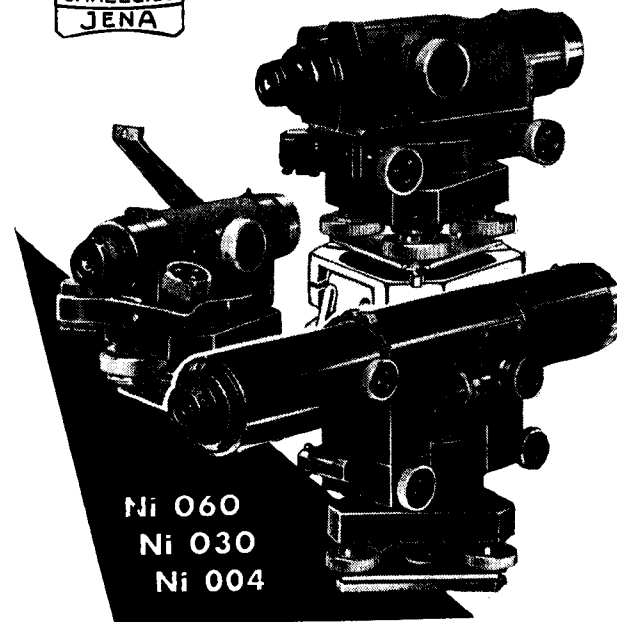


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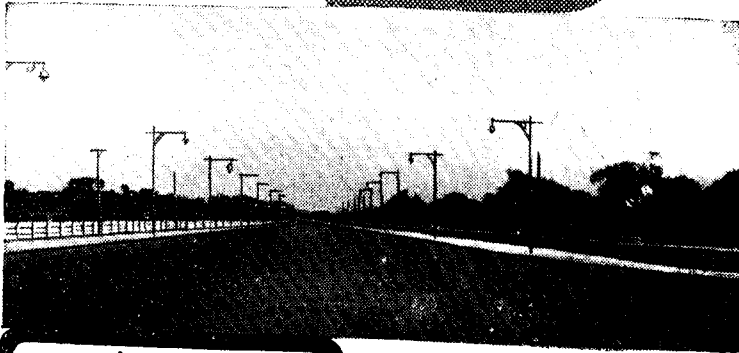
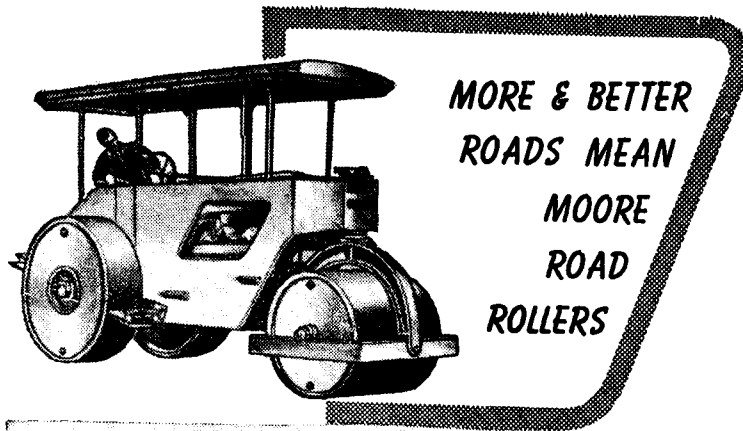
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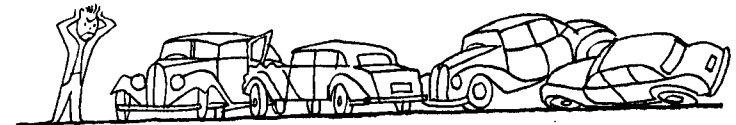
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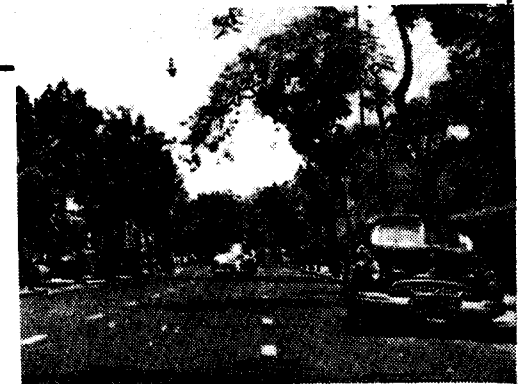
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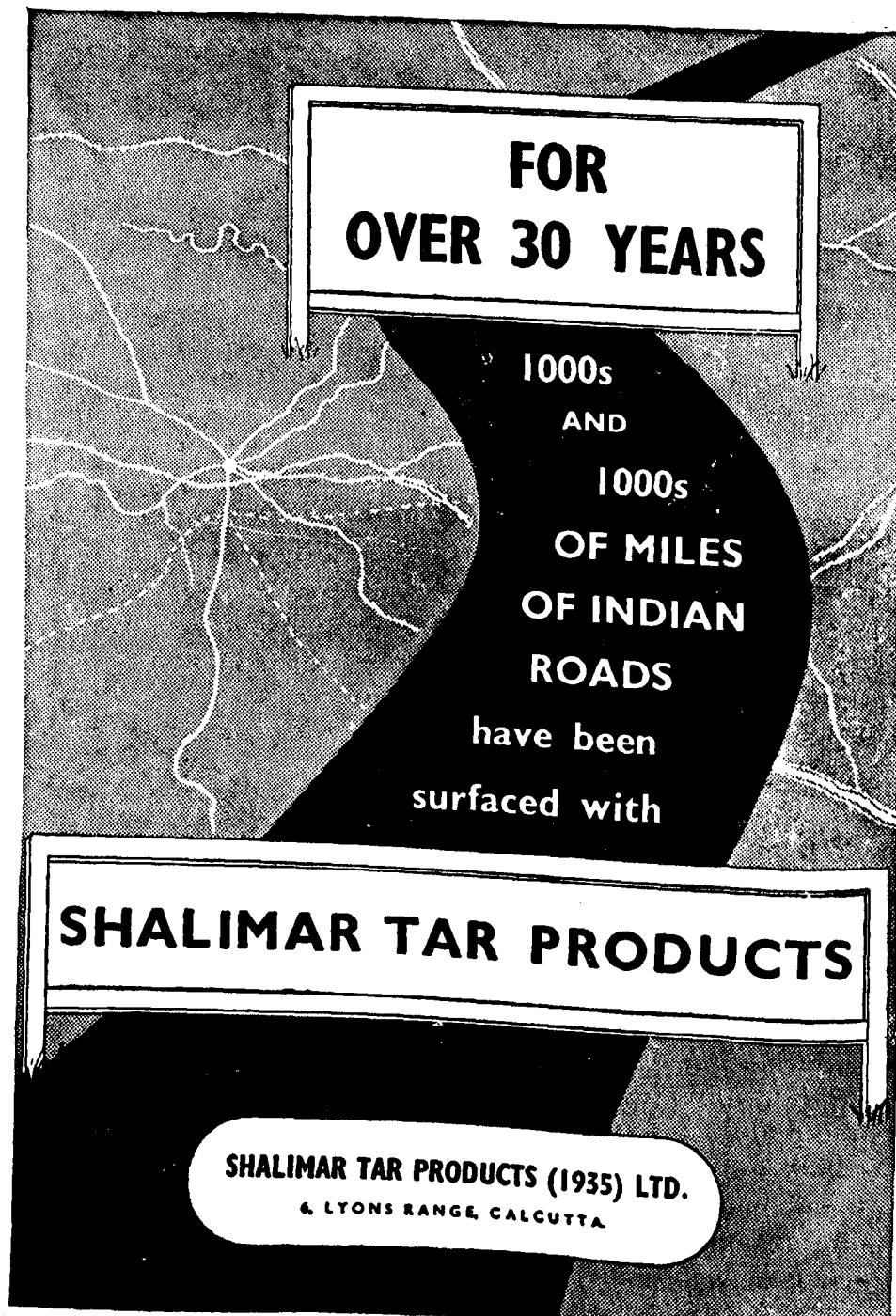
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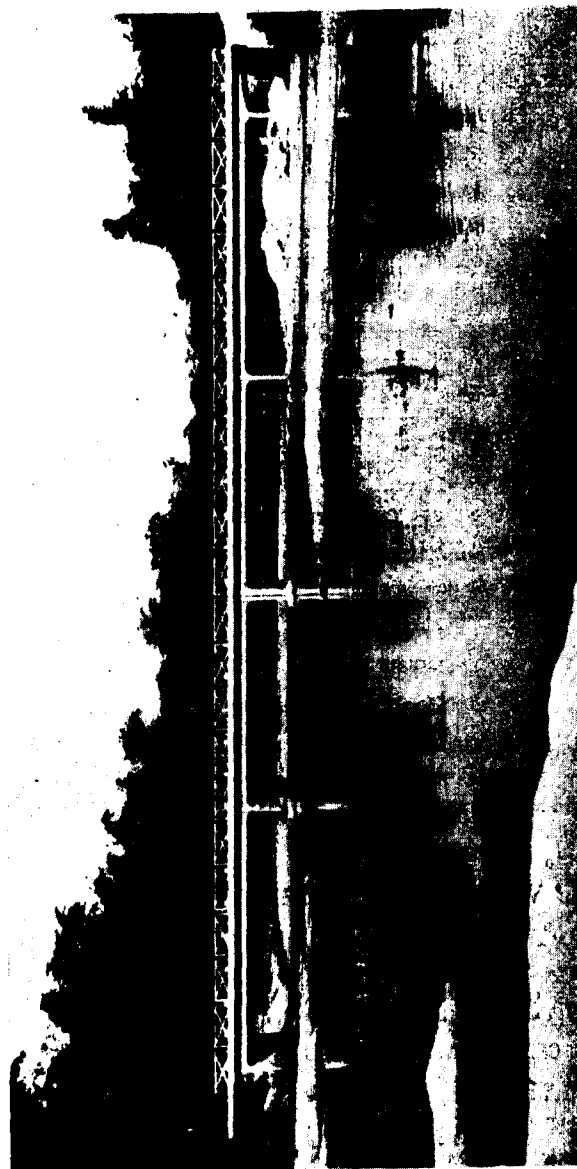


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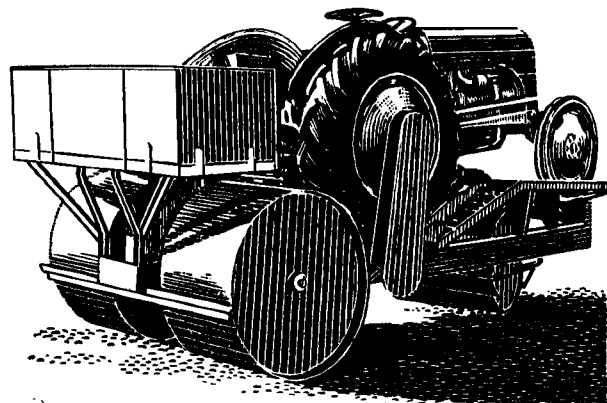
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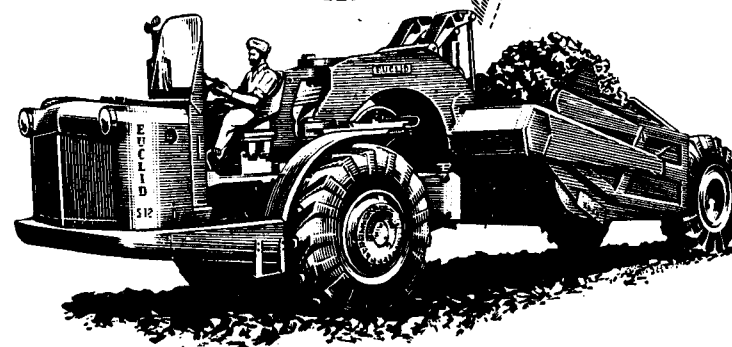
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