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Paper No. 233

"DESIGN AND CONSTRUCTION OF CEMENT CONCRETE PAVEMENT ON AGRA-MATHURA SECTION OF NATIONAL HIGHWAY No. 2"

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CONTENTS

SYNOPSIS

The Paper describes the various stages on design and construction of cement concrete pavement being laid on Agra-Mathura section of National Highway No. 2.

Determination of pavement thickness on the basis of '*k*' value and controlling wheel load has been discussed. Testing of materials, design of concrete mix and other relevant items connected with the construction have also been discussed.

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1. INTRODUCTION

1.1. The Agra-Mathura section of National Highway carries heavy traffic. An estimate for providing cement concrete pavement in this section was sanctioned in August, 1960, in order to cope with the anticipated increase of traffic. The work has been started some time back and is in progress at present. An Index Plan is given in Plate I.

1.2. The carriage width in this section of the road was widened to 22 ft about 4 years ago.

2. DESIGN OF CONCRETE SLABS ON AGRA-MATHURA ROAD

2.1. The following factors were taken into consideration while designing the concrete pavement:

(i) Amount, type and weight of present and anticipated traffic.

(ii) Supporting power and character of the subgrade.

(iii) Climate of the region in which pavement is to be constructed.

(iv) Strength and quality of concrete to be used.

2.2. Traffic Data

2.2.1. For determining the wheel load characteristics, traffic census was carried out on this road. The data collected in this traffic census is given in Table 1. Seventy per cent of this traffic was assumed to pass on one lane. It has also been assumed that the intensity of traffic in next 30 years will be double of that of the present traffic. The number of wheel loads thus evaluated under various groups are given in Appendix 1.

2.2.2. The designed life of the pavement is taken as 30 years or $30 \times 365 = 10{,}950$ days. This figure when multiplied by the value of wheel loads under various wheel load groups gives the total number of load repetitions of each wheel load group which are anticipated during the designed life of the pavement. The number of load repetitions per day that will produce 100,000 repetitions during the designed life of the pavement comes to $\frac{100{,}000}{30 \times 365} = 9.1$.

TABLE 1

Traffic census data

(Agra-Mathura Road)

(June 1961)

Sl. No.	Location mile	Average number of commercial vehicles (both directions): Loaded trucks	Unloaded trucks	Loaded buses	Unloaded buses	Average tonnage of trucks and buses	Total average tonnage of all pneumatic and non-pneumatic vehicles
1	Mile 756/6	164	73	138	26	1,954	2,825
2	Mile 771/3	155	33	103	3	1,620	1,841
3	Mile 781/2	121	40	121	12	1,489	1,775
	Average for the section	147	49	121	14	1,688	2,048

Total No. of commercial vehicles = 331

Note:—For calculating total tonnage, following laden weights have been assumed as per U.P., P.W.D., practice for traffic census:

Loaded trucks—7 tons; Unloaded trucks—2 tons; Loaded bus—4½ tons; Unloaded bus—1½ tons; Car—1 ton; Other vehicles (non-pneumatic)—½ to 1½ tons.

2.2.3. It is observed that the number of load repetitions per day is higher than 9.1 in all cases. The controlling wheel load, which is taken as the average of heaviest 100,000 of the anticipated wheel loads coming over the pavement, comes under group wheel load of 9,000 to 10,000 lb (being 8,800 lb). Taking the maximum of this group with 20 per cent impact, the controlling wheel load comes to 10,000 lb + 20 per cent = 12,000 lb, Appendix I.

2.3. Subgrade

2.3.1. In order to examine whether the soil in this section of the road conforms to the standard specifications, tests were carried out on the soil samples taken out from the subgrade in various miles, Table 2.

2.3.2. For determining modulus of subgrade reaction 'k', the bituminous coat on top was removed and stone water-bound maca-

TABLE 2

Showing results of physical tests on soil samples from Agra-Mathura Road

Sl. No.	Location mile	Atterberg's limits L.L.	P.L.	P.I.	Material retained on 200 mesh per cent	P.R.A. Group Classification
1.	768		Non-plastic		75	A-3
2.	768	18.4	17.0	1.4	62.5	A-4
3.	769		Non-plastic		73.4	A-3
4.	769	24.5	17.3	7.2	39.7	A-4
5.	770	18.1	15.7	2.4	63	A-4
6.	770	19.0	17.0	2.0	64	A-4
7.	771	20.5	14.7	5.8	24.6	A-4
8.	771	20.2	15.5	4.7	24.3	A-4
9.	772		Non-plastic		71.4	A-3
10.	773		Non-plastic		71.1	A-3
11.	773	19.0	16.4	2.6	43	A-4
12.	774		Non-plastic		—	A-3
13.	776	21.5	16.3	5.2	52	A-4
14.	778		Non-plastic		57	A-3
15.	779	21.3	16.8	4.5	34.2	A-4
16.	779		Non-plastic		68	A-3
17.	780	19.3	16.2	3.1	42.3	A-4
18.	780	17.0	13.2	3.8	58	A-4
19.	781		Non-plastic		74	A-3
20.	781		Non-plastic		70	A-3

dam surface was exposed. A very fine layer of sand was put over the exposed surface, over which a 12-in. diameter steel plate was placed and adjusted true to level. A jack, connected to a pump by a flexible pipe, was then placed on top of the plate. For obtaining reaction of the plate, two trucks were arranged, which were placed

back to back with some spacing. These trucks were loaded with jute bags filled with earth. A frame consisting of steel joists or steel trusses was attached to the back of the trucks and the centre of the frame was touching the top of jack in such a way that when jack was operated by pump and lifted, the frame was also lifted which in turn was held down by the loads of the trucks on both sides and the reaction was thus transmitted to the plate.

2.3.3. A datum frame was put on the ground adjacent to the jack to which dial gauges were fitted. These dial gauges rested on the plate to record settlements.

After initial adjustment, loads were applied at regular intervals and corresponding settlements recorded. Some of the results obtained are given in Fig. 1.

2.3.4. The load intensity (P in lb per sq. in.) which caused 0.05 in. settlement was determined and the 'k' value was given by $P/.05$. This value of 'k' was for 12 in. diameter plate.

2.3.5. For design purposes, 'k' value corresponding to 30 in. diameter plate is used. A correction factor was, therefore, applied. It has been observed[2] that 'k' value for 30 in. diameter plate is about half of the 'k' value for 12 in. diameter plate. The results of 'k' value for 30 in. diameter plate are given in Table 3. It will be observed that the 'k' value of the subgrade varies from 495 lb per sq. in./in. to 930 lb per sq. in./in. at four locations. At two locations, 'k' value of subgrade was evaluated after applying correction for saturation with the help of C.B.R. test and the value came to about 320 lb per sq. in./in. which is considered satisfactory. The possibilities of lowering of 'k' value due to change in moisture content are remote as the water-table is quite low and the soil is not clayey. For a concrete pavement, the 'k' value after correction for adverse moisture content should not be less than 150 lb per sq. in./in.

2.3.6. The 'k' value on water-bound macadam in the central portion (which is old) was found to be 1,120 lb per sq. in./in. The crust consisted of stone water-bound macadam and kankar with bituminous carpet or surfacing coat on top. In the widened portion (on sides), the 'k' value on water-bound macadam was found to vary from 510 lb per sq. in./in. to 1,180 lb per sq. in./in. The crust was made up of premix carpet, stone water-bound macadam and brick soling. The 'k' value was also determined on the recently consolidated water-bound macadam surface on a super-elevated portion of a curve and the value was found to be 872 lb per sq. in./in. It is considered that the minimum 'k' value of existing crust should not be less than 500 lb per sq. in./in.

Test No.	Location	Reading of proving ring for 0.05 in. deformation	Load in lb.	$K = \frac{P \times 20}{113}$	K (30)
I	Mile 780/4 W.B.M. edge	98	7684	1360	680
II	W.B.M. centre	157	12656	2240	1120
III	Subgrade on edge	77	5989	1060	530
IV	780/2 W.B.M. edge	72	5763	1020	510

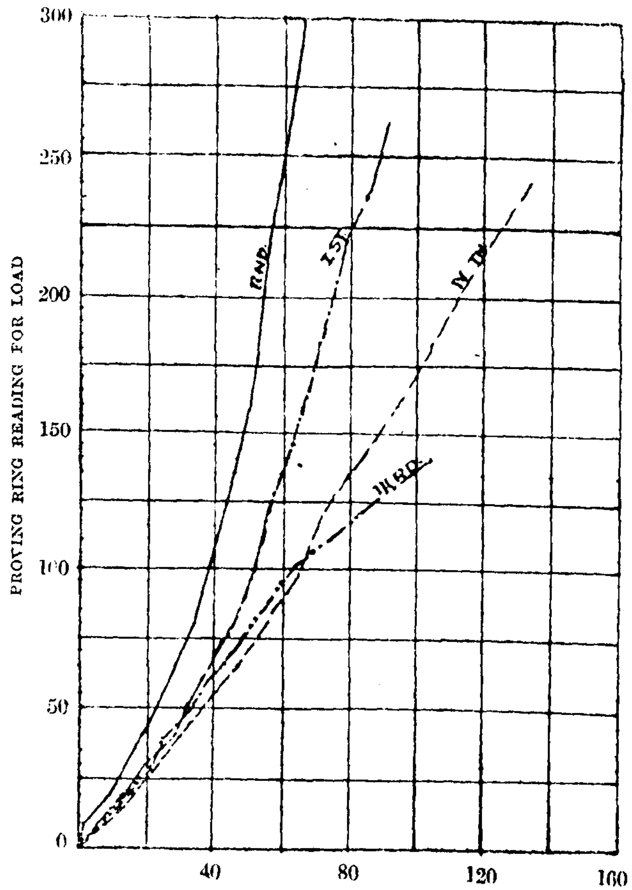

Fig. 4. Graph showing load-deformation curves for 'k' values

TABLE 3

Data about '*K*' values of different locations on Agra-Mathura section

Sl. No. of tests	Location and type of material tested	'*K*' value in lb/in.^{2}/in. settlement for 30 in. dia. plate	Dry bulk density of subgrade gm./cc	Moisture content of subgrade per cent	Thickness of crust (inches): Soling	W.B.M.	Surfacing	Total	Remarks
I.	**Original portion (central portion)**								
	Mile 780/4 (W. B. M.)	1,120	—	—	4½ in. (kankar)	3 in.	1 in.	8½ in.	
II.	**Widened portion**								
1.	Mile 770/8 W. B. M.	1,180	—	—	3 in.	6 in.	1¼ in.	10¼ in.	
2.	Mile 771/3 ,,	990	—	—	3 in.	6 in.	¾ in.	9¾ in.	
3.	Mile 771/5 ,, (near bifurcation)	1,000	—	—	3 in.	6 in.	1½ in.	10½ in.	
4.	Mile 779/2 W. B. M.	510	—	—	3 in. (brick)	5 in.	1 in.	9 in.	
5.	Mile 780/4 ,,	680	—	—	3 in. (brick)	5½ in.	1 in.	9½ in.	
6.	Mile 779/8 ,,	990	—	—	3 in.	4 in.	1½ in.	8½ in.	
7.	Mile 780/8 ,,	774	—	—	3 in.	4 in.	1 in.	8 in.	
III.	**Recently laid W.B.M. on curve**								
1.	Mile 781/1	872	—	—	3 in.	5 in.	2 in. (lean concrete)	10 in.	
IV.	**Subgrade**								
1.	Mile 780/4	540	1.67	9.05	—	—	—	—	Subgrade soil is sandy. '*K*' value of subgrade after applying correction for saturation with the help of CBR Test is 330 lb per sq. in./in.
2.	Mile 779/8	495	—	10.1	—	—	—	—	
3.	Mile 771/3	930	1.90	3.76	—		—	—	Corrected '*K*' value after applying correction, 310 lb per sq. in./in.

2.3.7. Tests on a few samples were carried out to determine shrinkage characteristics and these results indicated that lineal shrinkage varied from 0.5 to 3.0 per cent and volumetric shrinkage on the basis of wet soil varied from 8.6 to 15.8 per cent. The maximum permissible volume change in the soil is 17 per cent and lineal shrinkage 5 per cent.

2.3.8. The results of soil test, Table 2, indicate that the soils generally belong to A-3 and A-4 groups classification (P.R.A.) and are not of shrinkable nature.

2.4. Climate

2.4.1. The temperature variation, however, is considerable being 112°F. in summer and 36°F. in winter. These have to be considered in fixing the length of slabs.

2.5. Stresses in Concrete Pavement

2.5.1. Concrete

2.5.1.1. Studies were made to determine the most economical strength of concrete to be used under existing conditions. The thickness of concrete pavement was determined for '*k*' value of 500 lb per sq. in./in., controlling wheel load of 12,000 lb (including impact) and different modulii of rupture ranging from 500 lb per sq. in. to 650 lb per sq. in.[1], Table 4.

2.5.1.2. Assuming cost of materials as existing at present, the total cost of materials for different thicknesses of concrete pavement per mile was calculated, and is also given in Table 4. From the Table, it would be seen that the cost of concrete pavement per mile does not vary very much with different thicknesses when using mixes possessing different flexural strengths. For the purpose of the design described in this Paper, a flexural strength of 550 lb per sq. in. has been adopted.

2.5.2. Stress computation

2.5.2.1. When a corner of a pavement is curled upwards because of differentials in moisture content and temperature between the top and bottom of the slab, the support of the subgrade is reduced. Dr. Gerlad Pickett has developed semi-empirical formulae[1] for the above condition by assuming that subgrade support is lacking for some distance from the corner when a slab corner is curled upward. Design chart in Fig. 2 gives a solution of the above formula for protected corners.

TABLE 4

Showing cost of materials for different thicknesses of concrete road for different flexural strengths

Sl. No.	Compressive strength psi.	Flexural strength psi. (approx. 9.5 per cent of comp. strength)	Thickness of concrete slab designed on the basis of Portland Cement Association chart	Mix proportions by fineness modulus method (by weight)	Water-cement ratio	Total* cost of materials per 100 cu. ft. of concrete Rs	Cost of materials per mile of road Rs
1.	5250	500	8 in.	1:2.25:3.25	0.50	221.60	1,71,600 app.
2.	5775	550	(7.55) 7½ in.	1 : 2 : 3.0	0.47	237.30	1,65,500
3.	6300	600	(7.2) 7¼ in.	1 : 2 : 2.50	0.43	239.00	1,68,000
4.	6825	650	(6.75) 6¾ in.	1:1.75:2.25	0.39	252.40	1,65,300

****Note*** :—(a) The following costs of materials have been adopted:

1. Cement ... Rs 6.75 per bag (112 lb)
2. Sand ... Rs 80 per 100 cu. ft.
3. Stone ... Rs 90 per 100 cu. ft.

(b) Cost of labour and other items have been assumed to be the same for all the mixes.

2.5.3. Design of thickness

2.5.3.1. Concrete cubes and beams were cast for preliminary trials. The test result of these beams and cubes using the mix designed for 4,000 lb per sq. in. minimum compressive strength (5,400 lb per sq. in. designed strength) was found to give a flexural strength of 550 lb per sq. in. in 28 days. It was, therefore, decided to adopt flexural strength of 550 lb per sq. in. at 28 days for design purposes. This gives a working stress of 270 lb per sq. in. with a safety factor of 2. For a controlling wheel load of 12,000 lb and '*k*' value of 500 lb per sq. in./in., the design chart, Fig 2, gives a pave-thickness of 7.55 in. Thus, 7½ in. thick concrete slab has been provided. In practice, the thickness of concrete slab will vary from 7½ to 7¾ in. due to irregularities in the surface.

2.5.4. Fatigue resistance

2.5.4.1. As traffic data available was not sufficient, the total percentage of fatigue resistance of the designed thickness that will

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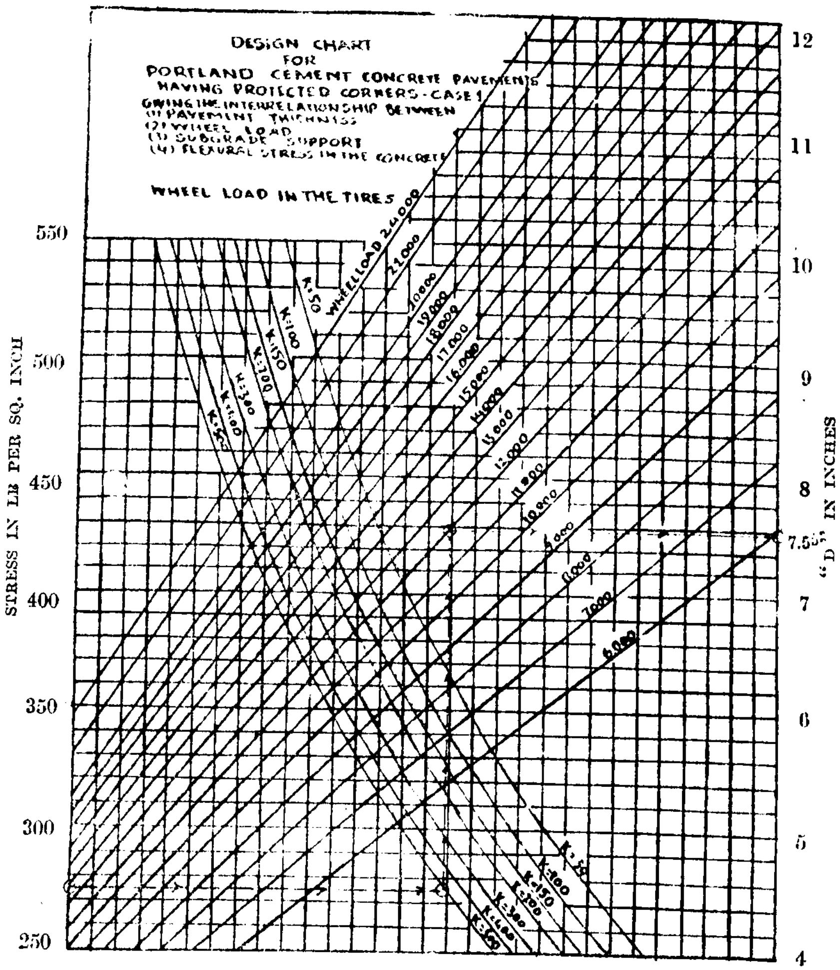


Fig. 2

be consumed by wheel load groups (having a safety factor less than 2), during the designed life of the pavement could not be evaluated. Theoretically, according to Portland Cement Association, the limit of fatigue resistance consumed by the anticipated traffic could be 100 per cent. On all, except the most heavily travelled roads, it is safe to exceed this limit by approximately 20 per cent. The reasons for this are that not all the wheel loads in a group will be equal to the heaviest for that group as has been assumed, the concrete

throughout its life will have a strength well above 28 days strength used in the design and the fact that the load repetitions are distributed over a period of years with a considerable time for recovery between loads (especially between critical heavier loads) reduces the effect of fatigue.

2.5.4.2. The designed thickness of $7\frac{1}{2}$ in. on Agra-Mathura section has been evaluated on the basis of maximum of wheel load group 9,000—10,000 lb plus 20 per cent impact and hence it can be assumed that this thickness would prove adequate from traffic, strength of subgrade and fatigue considerations.

2.5.5. Strengthening of edges and corners

2.5.5.1. The stresses produced by loading are greater at edges and corners of the concrete slab than in the middle portion. Consequently, if a balanced design is to be obtained, the edges and corners must be strengthened.

2.5.5.2. There are several methods in use, viz., thicknening the outer edges of the road, constructing a concrete beam or sleeper under the slab at the side of the road, extending the slab under the kerb or extending the base beyond the outer edges of the slab.

2.5.5.3. Widening of the base to at least 12 in. beyond the edge of the concrete slab is recommended if kerbs are not used. Such a widening gives additional support to the slab edges and gives extra protection against changes in moisture content in the subgrade near slab edges.

2.5.5.4. Widening the existing road (22 ft wide) to 25 ft has been adopted in this project. 2 ft wide soling with flat brick 3 in. thick was provided on both sides, followed by 2 coats of local stone metal each $4\frac{1}{2}$ in. thick consolidated to 3 in. in a width of $1\frac{1}{2}$ ft on each side, thus providing a crust thickness of $8\frac{1}{2}$ in. to 9 in.

2.5.5.5. As the concrete slab is $7\frac{1}{2}$ in. thick, and is placed on a good foundation, it was not considered necessary to strengthen the corners by providing hair-pin reinforcement[5].

2.5.6. Inter-face between slab and subgrade

2.5.6.1. The range of variation of temperature and its rate of change will be greater in a concrete slab than in the foundations and as a consequence, the slab will tend to slide over the foundation. This sliding will be restrained by friction between the slab and the base. To reduce the friction, the concrete slab was laid on the existing black-top surface on which irregularities were removed by laying lean concrete. The concrete slab thus came in contact in major portion of its area with the lean concrete surface. Efforts were made to make the lean concrete surface as uniform and smooth as possible. Use of waterproof paper or polythene film over the lean concrete surface would have reduced the friction but it would have been costly.

3. DESIGN OF JOINTS

3.1. Joints are either transverse or parallel to the longitudinal axis of the road. In order to be effective, they must :

(i) limit the effective length of the slab, thereby limiting the stress due to subgrade restraint ;

(ii) provide space for the slab to expand beyond their original length.

3.2. Expansion Joints

3.2.1. These joints control transverse cracking. They relieve compressive stresses that develop when a pavement expands by providing space into which slab ends may move. The expansion joints thus completely separate adjacent slabs, the space being filled with a compressible material and top of joint sealed with a sealing compound. The spacing of expansion joint adopted is 120 ft when constructed in summer and 90 ft when constructed in winter, i.e., when shade temperature is less than 20°C. The joints are 1 in. wide.

3.2.2. Dowel bars

3.2.2.1. Field experience shows that dowel spacings should neither exceed 18 in. nor be less than 8 in. Further, the clearance between outside dowels and pavement edge should be within the range of 4 in. to 8 in.

3.2.2.2. From Fig. 3, for $K=500$ lb per sq. in./in. and $d=7.5$ in., the length and spacing of a $\frac{3}{4}$ in. diameter dowel have been found to be 13 in. and 8 in. respectively. These are based on $f_s=30{,}000$ psi. and $f_c=1{,}800$ psi. The length depends on ratio of $\sqrt{\frac{f_s}{f_c}}$. As $\frac{3}{4}$ in. diameter dowel bars of mild steel would be used, f_s would be 18,000 psi. The permissible bearing pressure on 1 : 2 : 4 concrete can be taken as 450 psi[7]. Thus the length of $\frac{3}{4}$ in. dowel bars in this case would be

$$= 13 \text{ in.} \times \sqrt{\frac{1{,}800}{30{,}000}} \times \sqrt{\frac{18{,}000}{450}} = 13 \times \sqrt{2.4} = 13 \times 1.55 = 20 \text{ in.}$$

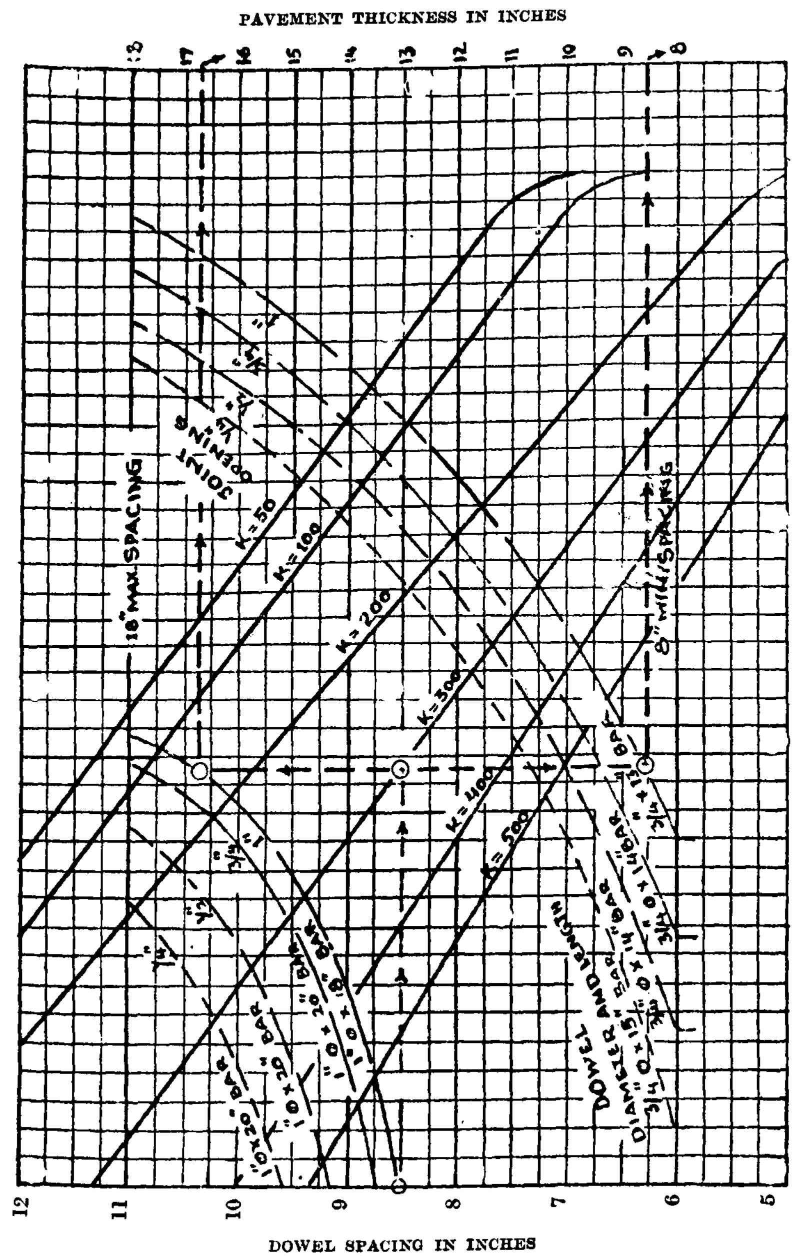
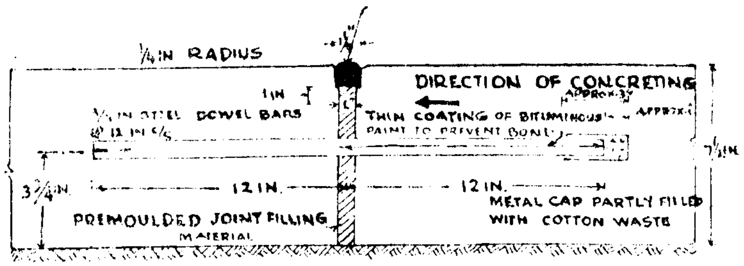


Fig. 3. Design chart for smooth round steel dowels used across expansion joints

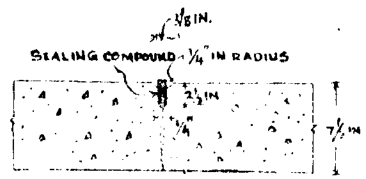
Actually ¾ in. diameter bars 24 in. in length have been proviedod at a spacing of 12 in. Details of joint are given in Fig. 4.

JOINT-SEALING COMPOUND

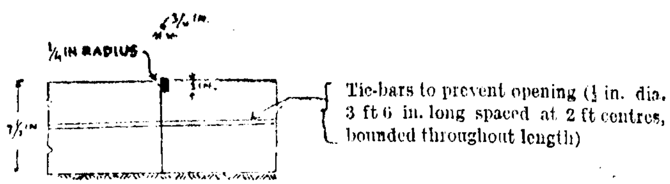


SUBGRADE OR BASE

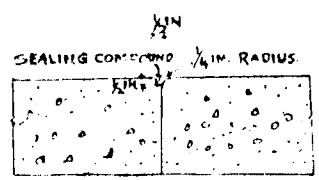
TRANSVERSE EXPANSION JOINT



TRANSVERSE CONTRACTION JOINT



TRANSVERSE CONSTRUCTION JOINT



LONGITUDINAL JOINT

Fig. 4. **Details of joints**

3.3. Contraction or Dummy Joints

3.3.1. Contraction joints in unreinforced concrete pavement are to be spaced at intervals which will control cracking. Local experience is the best guide for determining spacing. A spacing of 20 ft has been provided for slabs laid in summer and 18 ft for winter.

3.3.2. No dowels have been provided across the contraction joints. This has been done on the consideration that the slab lengths are short. Therefore even under conditions of maximum contraction only a very small joint opening will develop. The interlocking of the aggregate particles at the faces of the joint will provide a substantial and adequate amount of load transference, thus making the use of dowels unnecessary.

3.3.3. As stated elsewhere, 10 per cent lengths are being provided with reinforcement (wire fabric). But, at present, no consideration for increasing the spacing of contraction joints in reinforced slab is being made. Portland Cement Association suggests maximum spacing of 40 ft in such cases with dowelled contraction joints[1]. It is, however, necessary to ensure that no reinforcement should be carried through contraction joint.

3.3.4. A plane of weakness is induced by means of a groove as shown in Fig. 4. When subjected to tension, the slab will crack at this point and the crack will be permitted to open. The groove is to be kept filled with sealing compound to prevent any moisture entering the subgrade.

3.4. Longitudinal Joints

3.4.1. The longitudinal joints have been provided in the centre as the road width exceeds 15 ft. These joints prevent the formation of longitudinal cracks which will tend to occur in a slab of greater width.

3.4.2. No reinforcement has been provided across the longitudinal joints although some authorities recommend 'hinged joint' for this purpose by providing bonded steel to give relief for flexural stresses caused by curling of the slab. Plain butt joints have been provided as shown in Fig. 4.

3.5. Construction Joints

3.5.1. When unavoidable interruptions occur in the progress of work, a construction joint is introduced. It is, however, desirable that the day's work should finish at an expansion or a contraction joint.

3.5.2. Details of construction joints are given in Fig. 4, which provide a groove on top. Tie-bars $\frac{1}{2}$ in. diameter, 3 ft 6 in. in length, have been provided at 2 ft centres, bonded throughout the length.

3.5.3. Design of tie-bars

3.5.3.1. The tie-bars are used across the joints wherever it is necessary or desirable to ensure firm contact between slab faces or to prevent abutting slab from separating. These are designed to withstand tensile stresses only and not to act as load transfer devices. The maximum tension in the tie-bars across any joint is equal to the force required to overcome friction between pavement and subgrade from the joint inquestion to the nearest free joint or edge.[1]

3.5.3.2. Area of steel A_s in sq. in. required per ft length of joint $= \dfrac{bfW}{S}$

where b = distance between the joint in question and nearest free joint of edge in ft.

f = coefficient of friction between pavement and subgrade. (The value ranges between 1 and 2. It has been taken as 1).

W = weight per sq. ft. of slab in lb.

S = allowable working stress in steel in psi (18,000 lb per sq. in.).

In our case, $W = \dfrac{144 \times 15}{2} \times \dfrac{1}{12}$

$= 90$ lb

b can be 18 ft or 20 ft. Lower value has been adopted.

$$A_s = \frac{18 \times 1.0 \times 90}{18{,}000} = .09 \text{ sq. in.}$$

The spacing of $\frac{1}{2}$ in. diameter bar having an area of

$$0.20 \text{ sq. in.} = \frac{.20 \times 12}{.09}$$

$= 26.6$ in.

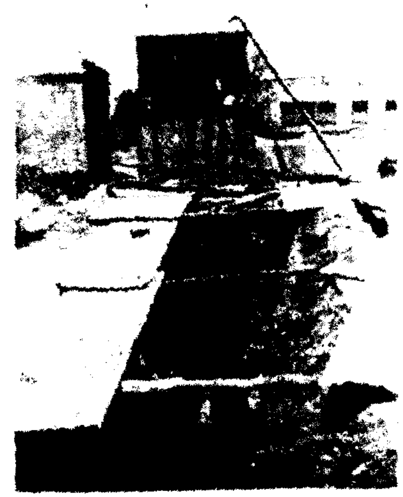


Photo 1

Showing installation of a pump in a pit to pump out water from well to the water tank

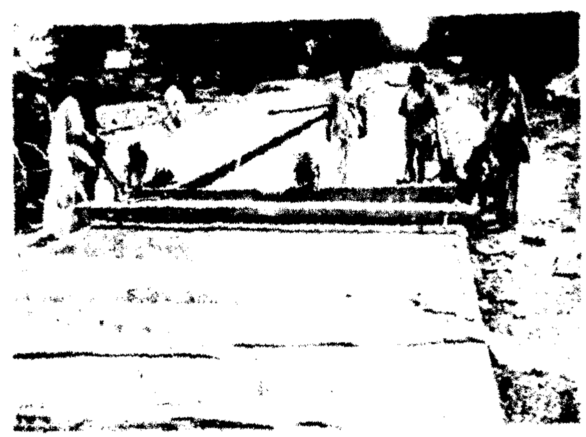


Photo 2

Compaction of the concrete pavement by screed-board vibrator

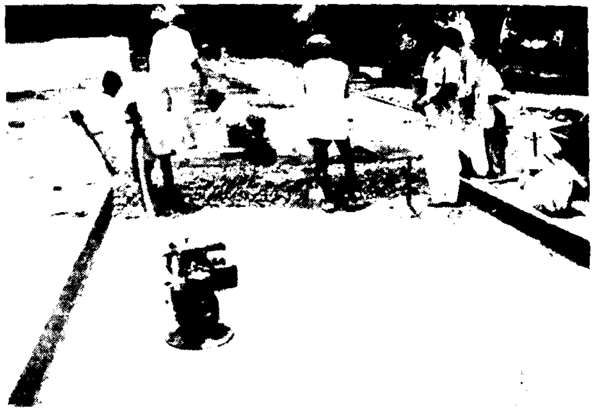


Photo 3

Compaction of concrete by poker vibrator at sides

Photo 4

Showing the concrete surface being finished by a brush. Also showing the fixing arrangements of channel forms

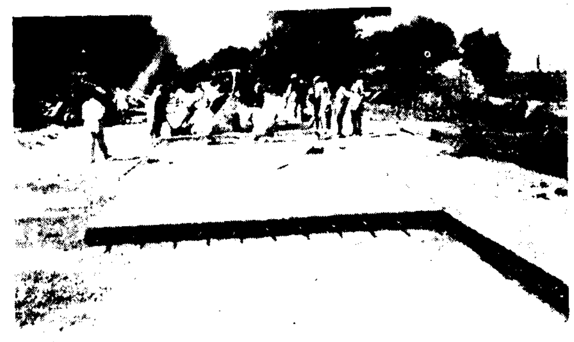


Photo 5

Showing dowel bars at expansion joint

Photo 6

Lifting the fillet from the expansion joint by a lever

Photo 7

The completed concrete road opened to traffic

3.5.3.3. The length of tie-bar should be at least twice that required to develop a bond strength equal to the working stress of the steel.

$$\text{Thus } L = \frac{2 \times S \times A}{f\,(\text{bond}) \times P}$$

where L = length of the bar in inches.

S = allowable working stress of steel in psi (18,000 psi)

A = cross-sectional area of one tie-bar in sq. in. (= .20 for $\frac{1}{2}$ in. diameter bar).

P = perimeter of tie-bar in in. (1.571 in. for $\frac{1}{2}$ in. diameter bar).

f (bond) = working stress in bond

= 100 lb per sq. in. (for 1 : 2 : 4 concrete)

$$\therefore\ L = \frac{2 \times 18{,}000 \times .20}{100 \times 1.571}$$

$$= \frac{7{,}200}{157} = 45.8 \text{ in.}$$

The actual length of tie-bar provided is 42 in. at a spacing of 24 in.

3.6. Design of Distributed Steel for Reinforced Slabs

3.6.1. Distributed steel is used in concrete pavements in the form of welded wire fabric or bar mats. Its principal function is to hold together the fractured faces of slabs after cracks have formed. Adequate load transmission across the crack is thus assured (by the interlocking action of the rough faces of the crack) and infiltration of incompressible materials into the crack is prevented. These fine cracks are harmless features of reinforced concrete roads and are not detrimental either to its performance or its life.

3.6.2. Reinforcement does not increase the flexural strength of an unbroken slab when used in quantities which are considered to be within range of practical economics. In unreinforced concrete, a crack can develop into a major defect through spalling of the cracked edges. Where, however, pavements are jointed in a manner that will control cracking, distributed steel has no function.

3.6.3. It is more economical to place joints at fairly close intervals and reduce the weight of reinforcement required.

2

3.6.4. Like tie-bars, the distributed steel is designed to withstand tensile stresses only. The maximum tension in the steel members across a crack is equal to the force required to overcome friction between pavement and subgrade from the crack to the nearest free joint or edge. This force will be greatest when the crack occurs at the middle of slab. Distributed steel is designed to withstand the tensile stresses when a crack develops at this location and for practical considerations, the same steel is usually used throughout the length of short slabs. On longer slabs (over 50 ft), saving can be realised by lighter steel for portions of slab near its ends[1].

3.6.5. Amount of longitudinal and transverse steel required per ft width or length of slab is :

$$A_s = \frac{LfW}{2S}$$

Where A_s = area of steel in sq. in. and L = distance in ft between free transverse joints when calculating longitudinal steel or between free longitudinal joints or edges when calculating transverse steel.

f = coefficient of friction between slab and subgrade.

The value ranged between 1 and 2 and has been taken as 1.0.

W = weight in lb of 1 sq. ft. of slab.

S = allowable working stress, psi of steel (as hard drawn steel wire is used in mesh reinforcement, this value is taken as 27,000 lb per sq. in.).

$W = 15/2 \times 1/12 \times 144 = 90$ lb per sq. ft.

A_s (longitudinal reinforcement)

$$= \frac{L.f.W.}{2S} = \frac{20 \times 1 \times 90}{2 \times 27{,}000} = .033 \text{ sq. in.}$$

$$A_{st} \text{ (transverse reinforcement)} = \frac{11 \times 1 \times 90}{2 \times 27{,}000}$$

$$= .018 \text{ sq. in.}$$

3.6.6. The most common type of mesh is 3in. × 12 in., spacing between (main) longitudinal bars being 3 in. and that of transverse bars 12 in. The reinforcement is provided in the form of sheets. Use of reinforcement supplied in rolls is not recommended. B.R.C. welded fabric (No. 109 as per B.S. 1221—45) provides S.W.G. wire No. 5 for main reinforcement and S.W.G. wire No. 10 for cross (transverse) reinforcement at above spacings. This gives a sectional area of longitutional wires as 0.14 sq. in. and sectional area of transverse wire as 0.013 sq. in. per ft width. Thus B.R.C. fabric No. 109 (B.S. 1221-45) is suitable. The weight of this fabric is 4.71 lb per sq. yd.

3.6.7. The reinforcement is preferably placed in single layer near top of slab as top reinforcement is easier to place and provides a greater insurance against crack opening in the surface than bottom reinforcement does. In addition, it is then easier to ensure that concrete near bottom of slab is fully compacted.

4. DESIGN OF CONCRETE MIX

4.1. Materials

4.1.1. Coarse aggregate

4.1.1.1. The samples of Faridabad (Delhi) stone, Bayana stone and Delhi-Punjab range stone were tested and the results are given in Table 5.

TABLE 5

Showing physical properties of stones

Sl. No.	Stone	Los Angeles abrasion value per cent	Aggregate impact value per cent	Aggregate crushing value per cent	Water absorption per cent
1.	Delhi-Punjab range	27.25	13.84	18.84	0.33
2.	Bayana stone	23.3	15.15	19.85	0.50
3.	Delhi (Faridabad)	34.5	17.65	21.25	0.70
	Specified limits	40	30	—	—

4.1.1.2. Results indicate that these stones conform to I.S. Specification 383—1952 and the specifications of cement concrete road construction[4] (i.e., the aggregate impact value not to exceed 30 per cent and Los Angeles Abrasion Test value not to exceed 40 per cent).

4.1.1.3. The maximum size of coarse aggregate used is 1½ in. Two sizes of coarse aggregates, viz., 1½ in. and ¾ in., have been collected separately in order to obtain a grading in the ratio of 75 per cent of 1½ in. size to 25 per cent of ¾ in. size by weight. The following grading limits were specified in contracts:

I.S. Sieve	Percentage by weight retained on I.S. Sieve		
	$1\frac{1}{2}$ in. size (contract bond)	$\frac{3}{4}$ in. size (contract bond)	$1\frac{1}{2}$ in. graded stone (after mixing 2 sizes)
$1\frac{1}{2}$ in.	0—5	0	0—5
$\frac{3}{4}$ in.	40—90	0—5	30—70
3/8 in.	100	0—60	65—90
3/16 in.	100	90—100	95—100

It was, however, felt that instead of collecting two sizes of stone, three sizes, viz., $1\frac{1}{2}$ in., $\frac{3}{4}$ in. and $\frac{3}{8}$ in., would have been collected in order to obtain better grading.

4.1.2. Fine aggregete

4.1.2.1. There are two possible sources of coarse sand, one from Chambal river and the other from Badarpur (Delhi). Due to greater distance involved, Badarpur sand is not available at the cost of Chambal sand. In Chambal sand also, fine and coarse varieties are available, the coarser one being quarried near Dholpur side.

4.1.2.2. Many samples of sand were tested for determining grading and fineness modulus. A typical grading obtained was as follows:

Details of sand		Percentage by wt. passing sieves						Fineness modulus	Silt & clay content	Organic impurities
	I.S. Sieve	3/16 in.	No. 240	No. 120	No. 60	No. 30	No. 15			
	B.S. Sieve	3/16 in.	7	14	25	52	100			
Grading zone 1 as specified		90–100	80–95	30–70	15–34	5–20	0–10	—	—	—
Grading zone 2 as specified		90–100	75–100	55–90	35–59	10–30	0–10	—	—	—
Chambal sand (typical grading)		98·94	96.58	90.80	77.90	16.84	1.50	2.17	2–3 per cent	Nominal
P.W.D. Specification (approx.)		95–100	85–95	55–85	30–60	10–30	0–5	—	—	—

4.1.2.3. It was stipulated in the specifications that "a fine aggregate whose grading falls outside the limits of any particular grading zone on sieve other than I.S. No. 60 by a total amount not exceeding 5 per cent shall be regarded as being within that grading zone. No tolerance is allowed for fine aggregate in any of the grading zones on the coarser limit."

4.1.2.4. It would be seen that although the sand used is quite suitable for concrete work, it is not complying with the grading limit specified for I.S. No. 60 (B.S. 25) sieve. This could not be helped as the sand is a natural sand and this deficiency is noticed in almost all samples tested. It is also seen that the concrete produced by using these aggregates is of good quality and as such no further improvement in grading of sand was considered necessary as that would have increased cost.

4.2. Water

4.2.1. Testing of water

4.2.1.1. Samples of water from wells, which would yield sufficient water,. were collected and tested.

4.2.1.2. The main difficulty lies in selecting water suitable for cement concrete work, when it contains some salts like sulphates or chlorides, etc., as there are no specified limits for the same.

4.2.1.3. A.A.S.H.O. Designation T-26-35—Standard method for test for quality of water to be used in concrete includes tests for (a) acidity and alkalinity (pH value), (b) total solids and inorganic matter, and (c) comparison of the given water with distilled water by making standard soundness, time of setting and strength tests with standard sand. It is specified that any indication of unsoundness, marked change in time of setting and a variation of more than 10 per cent in strength from results obtained with admixture containing the water of satisfactory quality shall be sufficient cause for rejection of water under test.

4.2.1.4. Abraham suggests tolerance upto 15 per cent in reduction of strength. Some authorities suggest following limits of soluble salts:

Sulphates—25 parts per 10^5 parts.

Chlorides—25 parts per 10^5 parts.

4.2.1.5. The results of tests on water samples are given in Tables 6 and 7 and they indicate that some wells contain water which have high percentage of soluble salts. These have not, therefore, been used for the cement concrete work.

TABLE 6

Showing results of chemical analysis of samples of water

Sl. No.	Location Mile	Total soluble salts parts/10^5 parts	Carbonate parts/10^5 parts	Bicarbonate parts/10^5 parts	Chlorides parts/10^5 parts	Sulphates parts/10^5 parts	Remarks
1.	766	76.40	Nil	32.00	11.44	Nil	
2.	767	1,290.6	3.6	55.00	508.37	160.00	
3.	768	46.80	Nil	26.49	8.56	Nil	
4.	769	62.60	Nil	40.66	15.72	4.40	
5.	770	120.80	4.85	62.84	36.18	8.22	
6.	771	227.80	3.64	61.61	99.96	5.70	
7.	772	168.00	3.03	19.15	77.59	6.20	
8.	773	646.60	2.42	78.85	309.40	Nil	
9.	774	236.60	12.12	118.29	81.40	Nil	
10.	775	1,235.60	Nil	4.93	550.05	58.07	
11.	776	243.9	4.85	54.22	94.25	Nil	
12.	777	288.6	29.09	110.90	106.08	Nil	
13.	778	2416.60	2.42	91.18	963.30	206.16	
14.	779	763.60	38.78	167.58	269.69	12.86	
15.	780	1,026.80	58.18	187.29	354.96	95.15	
16.	781	161.40	26.66	108.43	15.30	Nil	
17.	779/8	517.1	—	—	29.88	188.4	
18.	780/2	335.0	—	—	19.77	76.60	
19.	780/6	48.0	—	—	4.23	13.49	
20.	781/3	79.0	—	—	4.52	5.69	
							pH values
21.	771/1 (right side)	102.00	—	—	29.76	14.96	8.08
22.	771/3 (right side)	1,029.40	—	—	446.40	99.79	7.80
23.	771/Inspection House	336.10	—	—	148.80	33.95	7.80
24.	771/6 (left)	242.20	—	—	126.11	11.30	7.85
25.	771/7 (left)	304.50	—	—	131.32	11.84	7.50
26.	771/8 (left)	80.30	—	—	22.32	3.54	8.05
27.	771/9 (right)	124.70	—	—	41.29	16.23	8.00
28.	771/9 (left)	144.20	—	—	62.12	13.52	7.70
29.	772 (right)	145.50	—	—	176.70	45.99	8.52

Note :—Total soluble salts parts/10^5 parts (less organic matter) in samples at serials 17 to 20 are 514.6, 333.3, 44.08, 77.4 respectively.

TABLE 7

Results of physical tests on samples of water

S. No.	Location of water	Tensile strength of 1 : 2 cement—standard sand mortar at 8.4 per cent moisture psi		Compressive strength of 1 : 2 cement—standard sand mortar at 10 per cent moisture psi		Setting time in minutes		Soundness expansion	pH value	Remarks
		3 days	7 days	3 days	7 days	Initial	Final			
1.	Mile 779/8	326	332	2,202	3,269	145	231	1 mm.	7.95	Water is suitable
2.	,, 780/2	352	373	3,269	3,951	135	240	Nil	7.75	,,
3.	,, 780/6	355	421	2,486	4,264	103	243	Nil	7.80	,,
4.	,, 781/3	130*	188*	938*	1,290*	205	305	1 mm.	7.98	
5.	Tap water	323	380	2,064	3557	40	265	Nil	—	
6.	Distilled water	150*	209*	1,092*	1,067*	135	300	Nil		

Note :—*In case of water sample at Sl. No. 4 (mile 781/3), tensile strength and compressive strength of cement and standard sand mortar in ratio of 1 : 3 at 20 per cent moisture were determined and results compared with those with distilled water (Sl. No. 6). Reduction in strength is less than 10 per cent.

4.3. Design of Mix

4.3.1. The design of concrete mix was made earliar on the basis of minimum compressive strength of 4,000 lb per sq. in. at 28 days.

4.3.2. Cement, sand and coarse aggregate in ratio of 1 : 2 : 4 by weight, in which the ratio of coarse to fine aggregate was 3 to 1, and water-cement ratio of 0.50 was found to produce concrete of required strength (4,000 lb per sq. in. minimum in compression), workability (slump $1\frac{1}{2}$ in.), and compacting factor (.85 – .92).

4.4. Field Adjustment

4.4.1. Trial slabs were laid in the field with the designed mix as well as two other mixes which were 5 per cent leaner and 5 per cent richer. Cubes and beams were also prepared and tested. Results are given in Table 8.

TABLE 8

Results of preliminary tests on cubes and beams

Sl. No.	Details of specimens	Proportions of concrete mix (by weight)	Water-cement ratio	Compacting factor	28-day results: Average compressive strength psi	28-day results: Average flexural strength psi	Remarks
1.	Preliminary tests	1:2:4	0.50	0.945	5,800	575	
2.	Trial slab (field)	1:2:4	0.50	0.925	>6,400	540	
3.	Trial slab (field)	1:2:4 (5 per cent richer)	0.50	0.945	>6,400	460*	*One beam tested
4.	Trial slab (field)	1:2:4 (5 per cent leaner)	0.50	0.945	>6,400	500*	

4.4.2. While vibrating concrete under screed-board vibrator, it was observed that too much mortar was coming on top. It indicated that the mix required some adjustment. Adjustments were made at the site by gradually reducing sand content and increasing the coarse aggregate by the same weight. Ultimately it was found that mix proportion of 1 : 1.75 : 4.25 was quite workable and suitable for laying under screed-board vibrator. Formation of scum on top was also reduced. The water-cement ratio was kept

at 0.5. This mix is in a way economical as percentage of coarse aggregate is higher than in the original mix.

4.4.3. The results of cube and beam test on specimens cast at site indicated that very high compressive strength was obtainable even at 7 days. At 28 days also, the compressive strength was generally found to be higher than 5,000 lb per sq. in. and in some cases more than 6,000 lb per sq. in. The flexural strength was, however, lower than the minimum 650 lb per sq. in. which was originally aimed at. The flexural strength at 28 days interval was found to vary from 500 to 600 lb per sq. in. The usual relationship between compressive strength and flexural strength as noted in publications was found unworkable here.

4.4.4. The compressive strength and flexural strength results obtained on the same specimens at 28 days were plotted to determine their relationship. The results are shown in Fig. 5. These

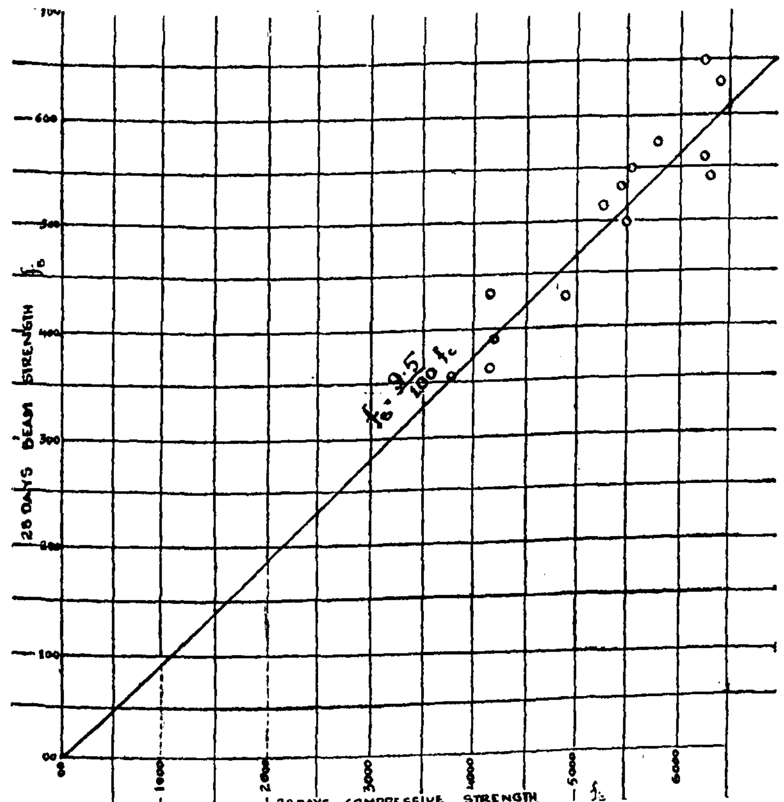


Fig. 5. Relationship between flexural and compressive strength of concrete

show that flexural strength is generally 9.5 per cent of compressive strength.

4.4.5. Unless, therefore, richer mix was used, flexural strength of the order of 650 lb per sq. in. at 28 days was not obtainable.

4.4.6. It was then decided that flexural strength of not less than 550 lb per sq. in. may be aimed at with this value, the thickness of concrete pavement was coming to nearly $7\frac{1}{2}$ in. which has been adopted.

4.4.7. Thus the mix proportions 1 : 1.75 : 4.25 which gave 550 lb per sq. in. flexural strength and more than 4,000 lb sq. in. compressive strength in preliminary trials was tentatively adopted.

4.4.8. The proportions of design mix were also checked on the basis of maximum density of aggregates. $1\frac{1}{2}$ in. size and $\frac{3}{4}$ in. size aggregates were mixed in different proportions and density was determined in accordance with Indian Standards Specification No. IS 383-1952. The results indicated that 75 per cent coarse aggregate of $1\frac{1}{2}$ in. size and 25 per cent of $\frac{3}{4}$ in. size gave density quite comparable to that obtained on the aggregate containing 50 per cent $1\frac{1}{2}$ in. size and 50 per cent $\frac{3}{4}$ in. size stone. Density was also determined after mixing coarse sand with the coarse aggregate. Results are given in Table 9. These also indicate that mixing coarse sand with coarse aggregate in ratio of 1.75 : 4.25 when coarse aggregate consisted of $1\frac{1}{2}$ in. and $\frac{3}{4}$ in. size in ratio of 75 : 25, gives maximum density.

TABLE 9

Showing density of combined aggregate mixed in different proportions

Sl. No.	Coarse aggregate $1\frac{1}{2}$ in. size per cent	Coarse aggregate 3/4 in. size per cent	Combined coarse aggregate and sand ratio	Density lb per cu. ft.	Voids per cent
1.	50	50	—	100.98	—
2.	60	40	—	101.86	—
3.	75	25	—	99.88	—
4.	75	25	2 : 1	116	24
5.	75	25	4.25 : 1.75	127	19
6.	$66\frac{2}{3}$	$33\frac{1}{3}$	2 : 1	121	22
7.	$66\frac{2}{3}$	$33\frac{1}{3}$	4.25 : 1.75	123	5.20

5. MACHINERY AND EQUIPMENT

5.1. The project was proposed to be completed in two seasons by employing two units for the concreting work. The following machinery and equipment are considered sufficient for one unit:

Sl. No.	Name of equipment and machines	No. used and found just sufficient
1.	Weigh-batchers	2
2.	Concrete mixers (1 bag capacity 10/7)	5
3.	Vibrators—Poker-type	3
4.	,, —Screed-board	4
5.	Pumping sets	3
6.	Side forms	1,920 rft.

The numbers indicated above allow for breakdown on a scale actually experienced.

5.2. Weigh-batchers arranged were of single and double bucket types. The latter types suitable for feeding 2 mixers were found more useful.

5.3. Poker-type vibrators were used for vibrating concrete near the side forms as well as near the joints where screed vibrator was not effective. Further, when screed-board vibrator failed in hot weather, poker-type vibrators were used for vibrating concrete in the whole width.

5.4. Screed-board vibrators were used for vibrating 11 ft wide slab at a time. These were 12 in. in length. These vibrators driven by petrol engine stopped running at short intervals in summer season and gave lot of trouble. These breakdowns in vibrators were responsible for slow progress on many occasions.

5.5. Side Forms

5.5.1. The cement concrete slab to be laid was $7\frac{1}{2}$ in. thick. It was, however, not possible to get $7\frac{1}{2}$ in. deep channels as this is not a standard size. 8 in. × 3 in. channels were only available and were, therefore, used as forms for laying $7\frac{1}{2}$ in. slab. These channels were fitted with stake housings fixing arrangements indigenously, which proved quite effective. Each form was provided with housing at each end through which a sliding locking plate was driven to ensure proper matching up of form work both in line and level at each end. The details are shown in Photo 4.

5.5.2. According to specifications the top of forms should not vary from a 10-ft straight edge by more than $\frac{1}{8}$ in. at any point and side of the form by more than $\frac{1}{4}$ in.

5.6. Pumps and Water Arrangements

5.6.1. As already stated, the supply of water could be arranged from wells along the road side. The depth of water level from ground surface generally varied from 20 ft to 34 ft in Mathura district. It was, therefore, not possible to pump out water by installing a pump on the well. Consequently, pits had to be dug adjacent to the wells for placing the pump closer to the water level. A hole was made in the steining of well for taking the suction pipe through it. Water was pumped to a storage tank and from there taken to the site of the work by pipe line by gravity flow, Photo 1. Pipe line was laid all along the road with tap connections at regular intervals of 100 ft or so. This arrangement proved quite a success. In dry and hot weather (May-June), water had to be pumped from two wells simultaneously to meet the requirement.

5.6.2. A sump pump which is run by compressed air was arranged as a standby. A light petrol-driven pump was also fitted on a platform which was fixed inside the well at the proper depth and was also used for pumping water. In this case, a driver had to go down to start the engine. This arrangement was convenient only with light pumps.

5.6.3. The above arrangements made curing effective and no difficulty regarding adequate water supply was experienced.

6. DIVERSION OF TRAFFIC

6.1. It was anticipated that traffic would have to be diverted from a stretch of main road for about three months. It was, therefore, decided to acquire 40 ft wide temporary land, abutting permanent land. On this land, 18 ft wide temporary diversion road was constructed by dressing the subgrade, compacting it by roller and then laying 18 ft wide brick-on-edge pavement with overburnt bricks. In the beginning, blinding with local earth was resorted to which filled up the joints and pavement became quite stable. This type of diversion did not develop much unevenness. Connecting links were made with brick-on-edge or chequered plates to allow the traffic to come back to the main road without unnecessarily traversing the diversion road for a longer length than was necessary.

About 40 to 50 per cent of bricks from the old diversion could be used again in a new stretch.

7. LAYING OF BASE COURSE

7.1. Removal of Irregularities

7.1.1. It was observed that irregularities of the existing surface should be removed before laying the concrete pavement as otherwise excessive quantity of concrete would be consumed in filling up the irregularities.

7.1.2. Two alternative treatments were decided for removing the irregularities:

(a) When the thickness of the existing crust was about 9 in. everywhere, and the surface irregularities after patch repairs, when tested by scratch template, were not in excess of $\frac{1}{2}$ in. in 10 ft, no levelling course was given. Elsewhere, rectification was done by providing a W.B.M. course over the entire width of the surface to be concreted.

(b) A levelling coat of lean concrete (1 : 4 : 8) using local hard stone and coarse sand was laid to get a cross-section having 1 in 60 camber. Where the thickness of levelling coat required was less than $\frac{1}{2}$ in., no coat was given and concrete pavement laid on the bituminous surface.

7.1.3. Alternative No. (a) was adopted in case of curves where superelevation had to be provided. In other lengths, it was considered that providing levelling course of W.B.M. would be time-consuming.

7.1.4. Initially, it was feared that placing lean concrete or the concrete pavement on B.T. surface might not be advisable on account of possible softening or creep of bituminous coat under high temperature prevailing in this region. After careful consideration, it was felt that as existing B.T. surface had become quite old, there was no such possibility. Moreover, it was also felt that removal of black-top coat would disturb the W.B.M. as stone on top would definitely be loosened. This would mean compacting the W.B.M. surface before laying cement concrete slab.

7.1.5. Considering all these points, it was ultimately decided that levelling course of lean concrete be laid in straight reaches.

8. LAYING OF CONCRETE PAVEMENT

8.1. Setting of Form

8.1.1. The channel forms were set and fixed on the existing surface for laying 11 ft wide strip first on left side of the road. 16 in. long stakes were used for fixing the forms. It was found that length of stakes had to be shorter for fixing the central line of the forms due to greater resistance offered by old stone W.B.M. to driving of stakes. At the edges, longer stakes had to be used as these were generally driven into the freshly laid water-bound macadam.

8.1.2. Forms were set over a length representing two days' work in advance of the point where concrete was being placed.

8.1.3. The forms were generally removed after 12—24 hours of placing of concrete. It has been specified that "if air temperature is below 10°C. at any time during the 12-hour period from the time the concrete is placed, the form shall not be removed until 36 hours after placing of concrete".

8.2. Mixing

8.2.1. For this project, 10/7 capacity mixers were used. The number of mixers used at a time depended upon the length of slabs to be laid in one day.

8.2.2. Coarse aggregates, sand and cement were fed into weigh-batchers by manual labour. The position of weigh-batcher was so adjusted that the hopper of the weigh-batcher could discharge the materials into the hopper of the mixers. Double bucket type weigh-batchers were useful for feeding 2 mixers at a time. The concrete was mixed for 2 to 3 minutes. Measured quantity of water was added to each batch of concrete. Adjustment for moisture content in sand was regularly made. Sand was tested for moisture content two to three times a day.

8.2.3. Concrete from the mixers was discharged into hand-carts which were taken directly to the place of laying.

8.3. Placing and Compacting Concrete

8.3.1. Placing and spreading concrete

8.3.1.1. Continuous bay method was adopted for all sections. All concrete was tipped from carrying vehicle slightly in advance of the working face and transferred into position by shovels. This shovelling was equivalent to partial remixing of the concrete and eliminated any segregation which occurred when concrete was being transported or tipped.

8.3.1.2. Hand spreading was always done with shovels and not with rakes. Each layer of concrete was carried square across the width of the bay, Fig. 6. The object of this was to ensure that the initial degree of compaction given to the concrete during placing was as uniform as possible over the whole area. The uncompacted concrete was placed so that its surface was slightly higher than top of the forms, the amount of such surcharge depended on

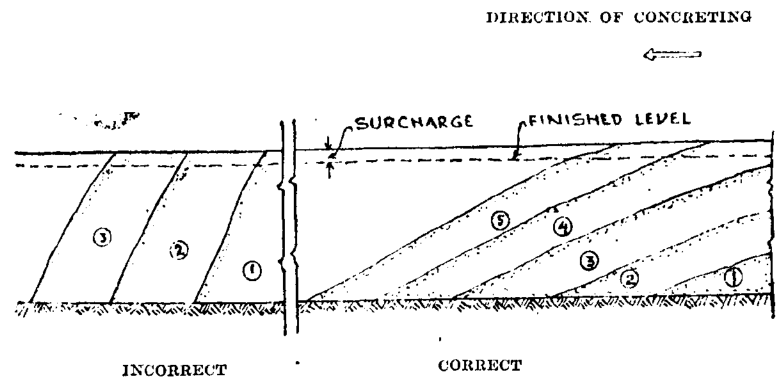


Fig. 6. Incorrect (left) and correct (right) method of spreading concrete by hand

the consistency of the concrete. It was found by experience that a surcharge of 1 in. to $1\frac{1}{4}$ in. was sufficient for the workability of concrete obtained at that particular site.

8.3.2. Compacting

8.3.2.1. Two types of vibrators were used, viz., screed-board vibrators, Photo 2, and poker (immersion) type vibrators.

8.3.2.2. Care was taken to use the beam of screed vibrator to spread concrete in such a way that as much compactive effort as possible was transmitted to the concrete. The vibrator having been laid across the surface of uncompacted concrete was allowed to sink down to form level under its own weight. After this strip had been compacted, vibrator was lifted up and moved forward about 1 in. less than its own width on to the next strip of uncompacted concrete and again allowed to descend to form level. Thus a fully compacted slab with a slight ridged surface was attained. Vibrators were never dragged along the form.

8.3.2.3. The compacting effort of hand-operated machine is much reduced towards the ends of the slab and this together with additional friction on the side forms, usually results in extensive honey-combing towards the edge of the slab, where the greatest strength is required. To overcome this trouble, poker-type vibrators with flexible shaft were used at the edges, (Photo 3). These were thrust into the concrete at short intervals along the slab.

8.3.2.4. During hot days, when petrol-driven screed vibrators stopped working, the poker vibrators were used to vibrate concrete in the main slab which was then compacted by hand tampers made of steel joist.

8.4. Methods of Measuring Compaction

8.4.1. Visual examination, though a valuable guide, is not enough for a quantitative comparison of different methods of compaction. Comparison of strength of cores cut and strength of cubes made from same mix or comparison of strength of cores taken from slabs compacted in different ways but made from same batch of concrete will not measure small differences. The best method so far achieved for assessing the degree of compaction seems to be the measurement of the variation in density from top and bottom of the slab.

8.4.2. Direct measurement of density

Cores of 6 in. dia. are drilled from full depth and each core is sliced into sections about 2 in. thick. Sections are dried to constant weight and weighed. Volume is also measured from which dry density is determined. If mix proportions are known, density of wet concrete immediately after compaction can be determined. Visual examination of a set of cores may also give valuable preliminary information on the degree of compaction achieved.

8.4.3. Degree of compaction on this project could not be determined as no facilities were available. Efforts are, however, being made to get a core drilling machine.

8.5. Finishing

8.5.1. The screed-board vibrator used for compaction was also used for finishing. No leading edge was provided. Three passes of screed were generally adopted. After laying concrete, a screed was passed over the forms which gave a surcharge of 1 in. to 1¼ in. Screed vibrator was then lifted in steps as described before, for compaction (first pass). Another similar pass was made. Before second pass, concrete in any portion found inadequate was made up by putting fresh concrete. After the second pass of screed vibrator, the vibrator was tilted slightly so that its leading edge was about 3/8 in. higher than the trailing edge. A slight sidewise oscillation was given to the vibrator beam as finishing proceeded. This helped to distribute the concrete along the front of the beam.

8.5.2. These vibrators were not allowed to run in one position for longer than necessary to compact or finish concrete, otherwise an excessive amount of fines would be brought to the surface where its presence was most undesirable.

8.6. Floating

8.6.1. After concrete was finished as stated above, the surface was further smoothed and consolidated by means of a longitudinal float 4 ft long and 3 in. wide, operated from a foot-bridge. The longitudinal float while held in a floating position was worked with a sawing motion parallel to the carriageway centre line and passed gradually from one side of the pavement to the other.

8.7. Straight-edging

8.7.1. After longitudinal floating had been completed and excess water had disappeared, but while the concrete was still plastic, the slab surface was tested for trueness with a 10-ft straight-edge swung from a handle 3 ft longer than one half the width of slab. This straight-edge was held in successive positions parallel to the road central line in contact with the surface and the whole area was gone over from one side of the slab to other. Advance along the road was in successive stages of not more than one half length of straight-edge. Any depressions found, were filled immediately with freshly mixed concrete, consolidated and refinished. High areas were cut down and refinished. Straight-edging and refloating was continued until the surface was perfect.

8.7.2. The slab surface was again tested for trueness with a 10-ft straight-edge and the wedge before the concrete began to set. This straight-edge was placed on the surface in successive positions parallel to the carriageway line. Irregularities were measured with the help of the wedge moved transversely at various points until it touched both the straight-edge and the concrete surface. At no point, the concrete did show a departure from the true surface by more than 1/8 in. If at any place the unevenness was greater than 1/8 in., not more than 3 passes of the vibrating machine were allowed and surface tested again in the specified manner. If the irregularity still exceeded this limit, the concrete was removed to a depth of 2 in. or to the level of the top surface of the reinforcement, if any.

8.7.3. The above procedure was adopted at each shifting of the straight-edge and the whole area gone over from one side of the slab to the other. Staight-edge advanced longitudinally in successive stages of not more than half the length of the straight-edge.

8.8. Belting

8.8.1. After straight-edging, when most of the water sheet had disappeared and just before the concrete became non-plastic, the surface was belted with a two-ply canvas belt not less than 8 in. wide and at least 3 ft longer than the width of the slab. The hand belts had suitable handles to permit controlled uniform manipulation. The belt was operated with short strokes transverse to the carriageway centre line and with a rapid advance parallel to the centre line.

8.9. Brooming

8.9.1. After belting and as soon as surplus water, if any, had risen to the surface, the pavement was given a broom finish with an approved steel or fibre broom not less than 18 in. wide, (Photo 4. The broom was pulled gently over the surface of the pavement from edge to edge, adjacent strokes being slightly overlapped. Brooming is done perpendicular to the centre line of the pavement and so executed that corrugations thus produced are uniform in character and width and not more than 1/16 in. deep. Brooming was completed before the concrete reached such a stage that the surface was likely to be torn or unduly roughened by the operation. The broomed surface had to be free from porous or rough spots, irregularities, depressions and small pockets such as might be caused by accidentally disturbing particles of coarse aggregate near the surface.

8.9.2. Trials were made by using glass plates having corrugations, in place of broom, for giving a final finish. This produced uniform corrugations on the concrete surface but the corrugations on glass plate wore out quickly by rubbing against concrete surface and frequent replacement of corrugated plates was needed. As arrangement for procuring replacement of such plates could not be made, brooming had to be resorted to which, however, did not produce such uniform corrugations.

8.9.3. After belting and brooming was completed, but before concrete became too hard, the edges of the slab were carefully rounded with an edger.

8.10. Curing

8.10.1. After the concrete had hardened, the slabs were kept covered with moist gunny bags for 24 hours. After that period, the bags were removed and the slabs were cured for another 21 days by holding water on them.

9. CONSTRUCTION OF JOINTS

9.1. Transverse Expansion Joints

9.1.1. 3/4 in. diameter steel dowel bars, 24 in. in length, were placed across transverse expansion joints at 12 in. centres in the middle of slab thickness, Fig. 4, Photo 5. The free or unbonded half of each dowel bar was painted with a thin coat of bitumen or bituminous paint applied cold. The film of bitumen extended across the joint gap to protect the dowel bar against corrosion. The free end of each dowel bar was provided with a metal cap, Fig. 4. The closed end of the cap was filled to a depth of 1 in. with soft compressible material (cotton waste) to provide space for the bar to move during expansion of concrete. The cap was a close sliding fit on the dowel bar. The edge of the cap was sealed during painting of the dowel bar, to prevent the cap from being filled with mortar.

9.1.2. Installation of joint

9.1.2.1. Device shown in Fig. 7 was used for concreting at the position of joints. This ensured setting of joint to the required grade and position.

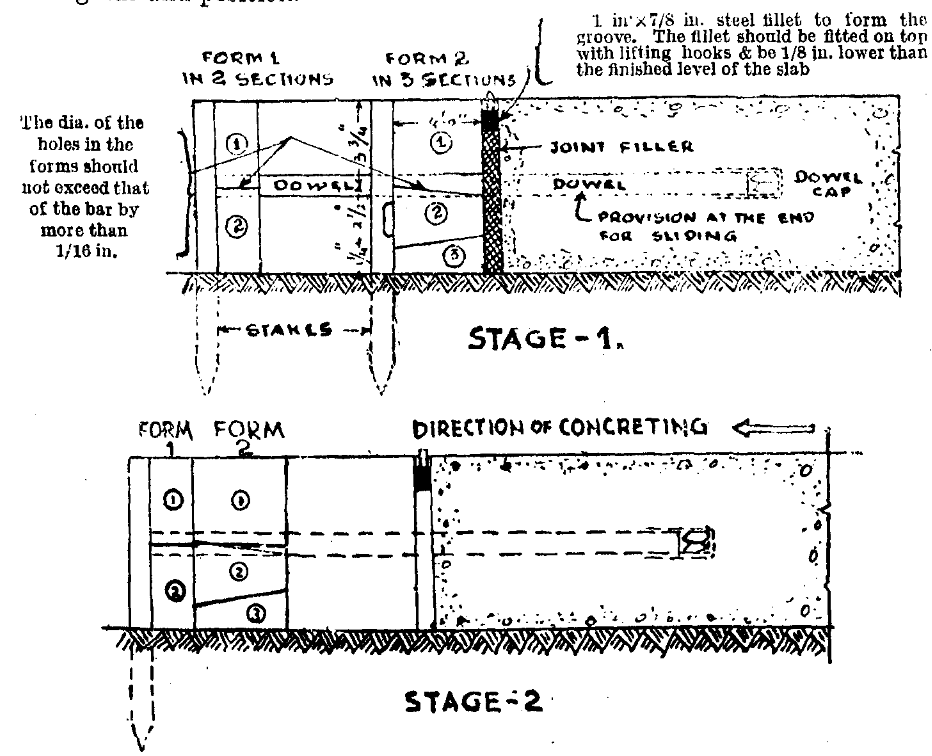


Fig. 7.

9.1.2.2. The premoulded joint filler, 1 in. thick, was put at the position of the expansion joint upto a depth of 1 in. below the surface of the pavement.

9.1.2.3. Concrete was deposited on the base as near to the joints as possible without touching them. Care was taken to see that load transfer devices were not disturbed in any way either before or after these were covered with concrete.

9.1.2.4. The concrete adjacent to the joint was compacted by means of internal vibrators. The vibrators were worked along the entire length.

9.1.2.5. The concrete was deposited and compacted first at the free end of the dowel bar upto the joint filler, Fig. 7. After about half an hour, the stakes supporting form 2 were removed and the three sections of this form were removed adjacent to form 1. Concrete was then placed and compacted between the joint filler and form 2.

9.1.2.6. Within about half an hour of completion of all finishing operations, including brooming on both sides of the joint and after the concrete had attained such a consistency as not to flow into the groove made by the fillet, Fig. 7, the fillet was carefully and slowly withdrawn by means of a lever, Photo 6, and the edges chamfered. The groove was temporarily sealed by putting a rope into it.

9.2. Transverse Contraction Joints (Dummy Joints)

9.2.1. The transverse contraction joints were provided by forming a groove in the top of the slab. The groove was 3/8 in. wide at the surface and 1/4 in. wide at the bottom and extended vertically downwards to a depth of $2\frac{1}{2}$ in., Fig. 4. These joints were located at 20 ft or 18 ft intervals depending on whether the expansion joint spacing was 120 ft or 90 ft.

9.2.2. Within about half an hour of completion of all finishing operations including brooming, and after the concrete attained such a consistency that it would not slump and close the slot, an oiled T-iron of 11 ft length and fitted with lifting hooks at both ends was forced into the compacted concrete. Soon after, the T-iron was removed carefully. In the groove so formed, an iron strip of same thickness as the T-iron and of breadth 1/8 in. less than the depth of the groove was temporarily inserted.

9.2.3. The concrete displaced during insertion of the T-iron was dispersed over a width of at least 12 in. on either side of the joint.

9.2.4. Special precautions were taken to ensure that the removal of the T-iron was not delayed unnecessarily; otherwise incipient cracks might appear during removal and these might later cause spalling.

9.3. Transverse Construction Joints

9.3.1. The construction joints were provided at the end of each day's work or where unavoidable interruption for more than 30 minutes occurred in concreting operation.

9.3.2. In the beginning when progress of work was slow or interruption occurred due to failure of machines, etc., the length of pavement between expansion joints could not be laid in one day and in that case construction joints had to be given at the end of day's work, but it was always ensured that these construction joints were constructed at the location of dummy joints. Specifications, however, permitted putting up such a joint so as to make a slab at least 10 ft in length.

9.3.3. These joints were plain butt joints with 1/2 in. diameter tie bars 3 ft 6 in. in length placed at 2 ft centres bonded throughout the length, Fig. 4. The exposed face of the joint was painted with bitumen before concreting the adjacent bay.

9.4. Pouring Joint Sealing Compound

9.4.1. After the curing period was over and before pavement was opened to traffic, temporary seal of all transverse expansion and contraction joints was removed and slots filled with joint sealing compound conforming to specifications, Appendix 2. The joint opening was thoroughly cleared of all foreign matter before the sealing material was placed.

9.4.2. The edges of joints were primed with a primer which was allowed to dry before the sealing compound was applied. The sealing compound was heated and laid at a temperature of 360 deg. F.

9.4.3. The sealing compound was poured into the joint opening in such a manner that the material did not spill on the exposed surface of the concrete. Use of lime-wash on exposed surface was found useful for this purpose. Any excess filler on the surface of the concrete pavement was removed immediately and pavement surface cleaned. The road was then opened to traffic, Photo 7.

10. REINFORCED SLABS

10.1. It is also proposed to observe the behaviour of reinforced and unreinforced slabs under identical conditions. Therefore, reinforced slabs have been laid in certain lengths.

10.2. The reinforcing steel was free from scale or other substances which might impair bond between concrete and reinforcement.

10.3. The reinforcement was placed 2 in. below top of concrete surface. In a 20-ft long bay, the length of bar mats was 19 ft 6 in., as the reinforcement was to be stopped short by 3 in. on either side of all transverse expansion or contraction joints. 6 in. overlap was provided transversely. The minimum distance from the edge of the slab to the longitudinal reinforcement was $4\frac{1}{2}$ in. On both sides of expansion joint, a strip of reinforcement fabric of similar gauge, 2 ft 9 in. in width, was placed immediately below the main reinforcement, with main wires parallel to the joint.

10.4. Laying of Concrete

10.4.1. In one method, concrete is placed in 2 layers, the initial layer is spread to a depth which after compaction will be 2 in. below the top surface of the pavement and over which bar mat will be placed. After placing the reinforcement, in position, the top course is poured, compacted and finished to the correct level.

10.4.2. The above method involved laying of concrete to a compacted depth of $5\frac{1}{2}$ in. in one full bay first before putting reinforcement. With the usual 1-bag capacity mixers and hand-screed vibrators, this may take time and before reinforcement is put and top layer of concrete laid, initial setting may start. This method, therefore, may not be practicable in summer season or when a full bay cannot be laid within the desired time.

10.4.3. In the other method, the reinforcement sheet is held in position, i.e., 2 in. below the top surface of concrete, by hanging the mat by means of hooks attached at short intervals to pipes placed transversely on channel forms. After this, concrete is poured from top which passes through the mesh. Compaction is usually carried out by poker and screed vibrators. As the concreting progresses, the hooks are disengaged from the reinforcement sheet and pipes shifted ahead to support other length of sheet. Both the methods were adopted in this project.

11. QUALITY CONTROL

11.1. A field laboratory was set up and equipment for testing cement, aggregate (fine and coarse), and concrete was arranged.

11.2. Tests on Aggregates

11.2.1. Grading tests (by sieve analysis) of fine and coarse aggregate were carried out regularly to find out whether any adjustment was necessary for achieving the specified grading. At least one test each on sand and stone ballast was done daily.

11.2.2. The percentage of moisture in aggregate was also determined regularly. In the beginning, 2 or 3 tests were done, particularly on sand, and correction for moisture was applied accordingly in the amount of water to be added for mixing concrete. The weight of aggregates in relation to their water content was also adjusted. Later on, test was made once a day. During summer, no moisture variation was observed in coarse aggregates as these were stacked in the open for sometime before being used. In case of sand, however, variation was observed. The quality of coarse aggregate was also checked by carrying out impact value test whenever any suspicion arose.

11.3. Tests on Cement

11.3.1. Concrete cubes and beams test results also indicated the quality of cement being used. Therefore, no separate test on cement samples was generally done.

11.4. Tests on Concrete

11.4.1. For checking strength, cubes and beams were regularly cast and cured at the site under similar conditions.

11.4.2. Specifications demanded casting of 12 beams and 12 cubes per day, i.e., 3 sets, each consisting of 4 beams and 4 cubes. One set was to be cast for 125—150 cu. ft. of concrete. The casting of beams and cubes was to be spread over the day's working period. This could not be followed on account of difficulties in sending so many cubes and beams for tests at Lucknow Research Laboratory. 4 cubes and 4 beams were cast daily. When facilities of testing cubes and beams at the project site would be available in near future, the number could be increased. Out of 4 beams and 4 cubes, one specimen of each type was got tested at 7 days and two at 28 days. The fourth specimen from each set was to be kept for further testing if and when considered necessary during the period of construction or after pavement showed sign of developing defects.

ACKNOWLEDGMENTS

The Authors express their thanks to Shri S. S. L. Bhatnagar and Shri Y. K. Gupta who as Asstt. Engineers in charge of the project took keen interest in planning and execution of the work. Thanks are also due to Shri S. C. Verma, Asstt. Engineer, who helped the Authors in designing the mix and carrying out tests.

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Appendix 1

DETERMINATION OF CONTROLLING WHEEL LOAD

The average number of different commercial vehicles passing through a section of Agra-Mathura Road (22 ft wide) in both directions is as follows: (Table 1)

Loaded trucks	*Unloaded trucks*	*Buses (Loaded)*	*Unloaded buses*
147	49	121	14

It is assumed that 70 per cent of these vehicles will ply on one lane as the drivers have tendency to go in the middle of road when there arch no other vehicles coming from opposite direction.

Traffic census carried out in 1957 revealed that average traffic intensity was 1,550 tons in mile 782. The average intensity now (1961) is 1,775 tons in mile 781. Thus, there is an increase of 15 per cent in 4—5 years' time. On this basis it can be assumed that in 30 years' time, the increase in traffic will be 100 per cent. Considering these two factors, the average number of commercial vehicles after 30 years would be :

Loaded trucks	...	397
Unloaded trucks	...	132
Buses (loaded)	...	327
Unloaded buses	...	38
Total	...	894

It has been assumed that 25 per cent of trucks are petrol-driven and 75 per cent diesel-driven. On the basis of laden and unladen weights of trucks and buses (information given by Transport Department), the axle loads, wheel loads and number of repetitions of the load per day are given below :

	No. of vehicles per day	Weight lb	Front axle load	Rear axle load
1. Petrol (trucks), unladen	100	8,400	2,800 (1,400)	5,600 (2,800)
2. Petrol (trucks), laden	32	24,900	8,300 (4,150)	16,600 (8,300)
3. Diesel (trucks), unladen	100	10,200	3,400 (1,700)	6,800 (3,400)
4. Diesel (trucks), laden	297	26,400	8,800 (4,400)	17,600 (8,800)
5. Buses, unladen	327	10,200	3,400 (1,700)	6,800 (3,400)
6. Buses, laden	38	17,400	5,800 (2,900)	11,600 (5,800)

Note **:—Figures in brackets are wheel loads taken as 1/2 of axle load.**

It has been further assumed that front axle takes 1/3 load and rear axle 2/3 load. It is seen from above Table that maximum axle load at present is 17,600 lb or say, 18,000 lb which gives a wheel load of 9,000 lb.

It will not be unreasonable to presume that wheel load in next 30 years would certainly increase and thus if wheel load of 10,000 lb with 20 per cent impact is assumed, it gives a controlling wheel load of 12,000 lb which has been adopted in the design.

Appendix 2

SPECIFICATIONS OF JOINT FILLER AND SEALING COMPOUND

Premoulded Joint Filler

It shall be of fibre board, impregnated with bituminous materials and shall comply with following requirements :

(i) It shall be able to withstand storage and handling without suffering damage.

(ii) It shall be easy to instal and easy to cut with normal working tools to exact sizes.

(iii) It shall be compressible but shall not extrude on compression.

(iv) It shall recover after release compressive force as to fill the joints completely at all times.

(v) It shall not deteriorate through fatigue.

The joint filler can be tested to conform to A.S.T.M. D-544-49. A sample of expansion jointing material to be used was got tested and following results were obtained :

1. Recovery 85.3 per cent (specified minimum 70 per cent). Compression was 206.4 psi (specified between 100 and 700 psi)
2. Loss in weight on compression—0.08 per cent (specified maximum 3 per cent).

Thus the sample was according to A.S.T.M. Specifications.

Joint Sealing Compound

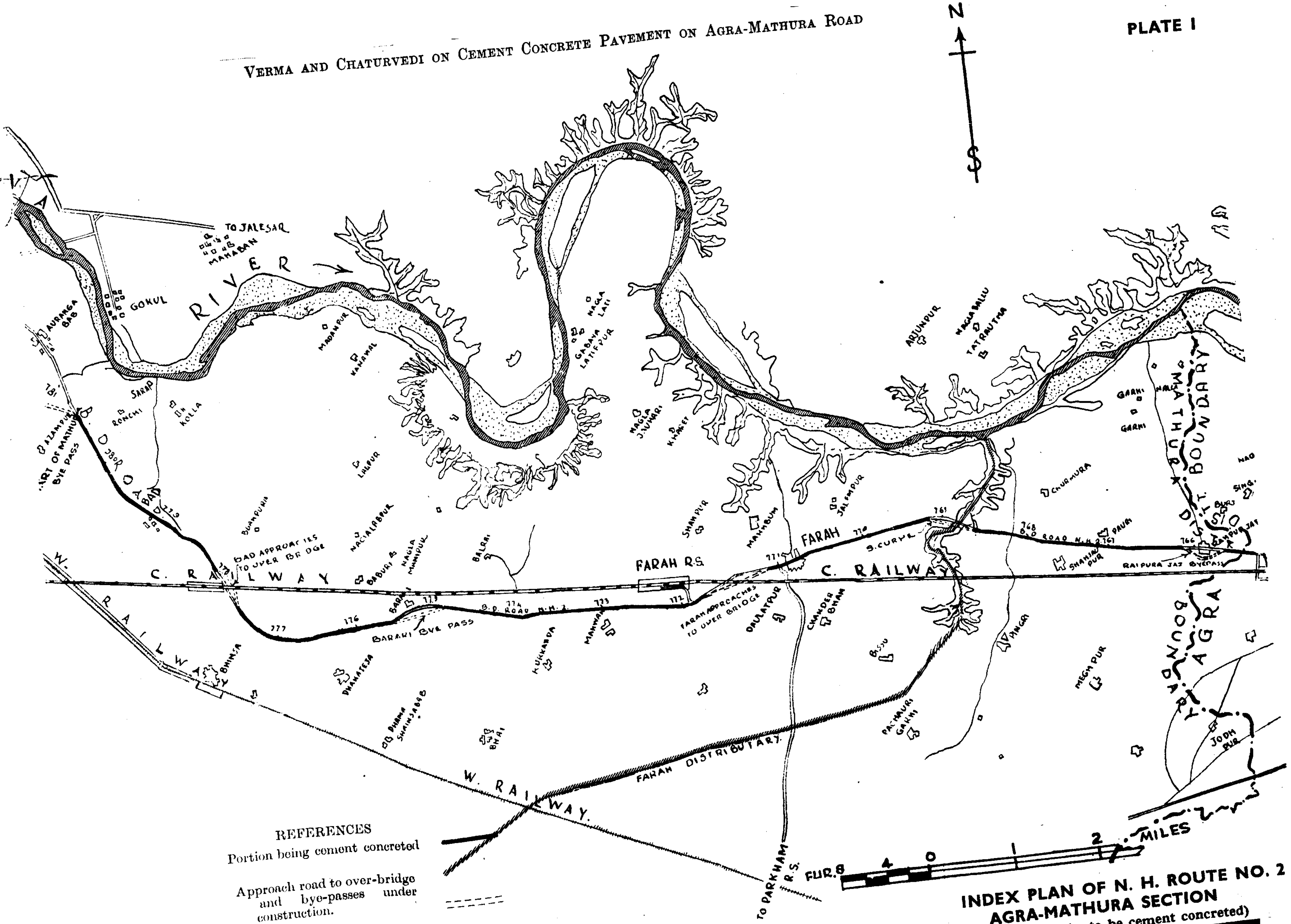
It shall consist of bitumen into which a small proportion of natural or reclaimed rubber with suitable plasticising oil and a mineral filler have been incorporated and shall possess the following properties :

1. It shall be easy to instal and shall not be susceptible to damage by heat or other causes during installation.
2. It shall adhere strongly to the concrete.
3. It shall not flow down into the joint nor across the joint from crown to kerb in hot weather.
4. It shall not become excessively brittle in winter nor so hard that it cannot accommodate itself to the movement of the joint.
5. It shall at all times resist the ingress of stones.
6. It shall fulfil its functions with minimum of maintenance.
7. It shall retain its plastic properties for a long period and shall not become brittle by loss of volatile oils nor as a result of ageing.

British Standard Specifications 2499—1954 on tests to assess the properties of hot applied joint sealing compound for concrete pavement provide basis for judging the quality of the compound to be used. The properties which can be determined are (a) pour point, (b) mineral filler, (c) flow, (d) resistance to grit, and (e) adhesion and extensibility.

For 1 *in.* thickness of joint, 4.5 gallons (.7 cu. ft.) of joint sealing compound is required for 1 in. depth and 100 running feet of joint. An allowance of 5 per cent for wastage should be made. The quantity of 1 gallon of primer is required for 1,200 running feet and 1 in. depth of joint.

PLATE I



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Portion being cement concreted

Approach road to over-bridge and bye-passes under construction.

INDEX PLAN OF N. H. ROUTE NO. 2 AGRA-MATHURA SECTION

(Showing lengths to be cement concreted)

Paper No. 234

"DESIGN AND CONSTRUCTION OF AJOY BRIDGE"

By

S. K. SAMADDAR*

&

A. K. ROY†

CONTENTS

SYNOPSIS

The Paper describes the salient features of design, and construction of a prestressed concrete bridge built over Ajoy river in mile 14 of the Panagarh-Illambazar-Suri State Highway. The bridge consists of 10 spans of 148 ft each with two-end spans 119 ft each. For facility of erection the prestressed girders were cast on the river bed in two sections, erected in position and then jointed.

1. INTRODUCTION

1.1. The bridge now under construction over the river Ajoy, is the longest prestressed concrete bridge so far built in West Bengal and possibly one of the longest prestressed concrete bridges in India.

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It is located in mile 14 of the Panagarh-Illambazar-Suri State Highway which branches off N.H.2 at Panagarh. The river forms the border of Birbhum district with that of Burdwan.

1.2. So far there has been no through road communication from Burdwan to Birbhum district and the only approach was rail link. In the fair weather, road communication is established by the construction of a temporary wooden bridge over the Ajoy river near the proposed bridge site. Due to shallow depth of water during dry season and because of high velocity of flow in the monsoon period, the establishment of a permanent ferry service to maintain through communication round the year was not possible. The absence of a permanent high level bridge was a serious bottleneck to the advancement of trade and commerce between industrially advanced area of Burdwan and the southern region including Birbhum district rich in agricultural products. Apart from this, the construction of other bridges will provide easy travel to the places of religious and cultural importance.

2. GENERAL FEATURES OF SUPERSTRUCTURE

2.1. The general arrangement of the spans of this bridge is shown in Fig. 1 and plate I. In this arrangement, the left end of the girder No. 1 rests on the left abutment with the other end cantilevering out 30 ft beyond pier No. 1. Girder No. 2 rests at one end on the cantilever arm of the first girder and extends 30 ft beyond the pier No. 2 and in the same way as the first girder. Similar arrangements are repeated from both the abutments and linked in the middle by a suspended span.

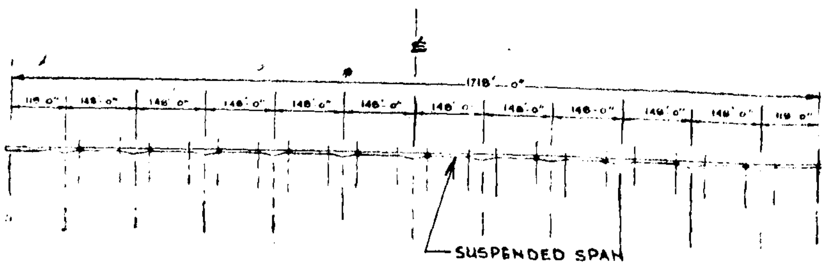

Fig. 1 (a). **During service**

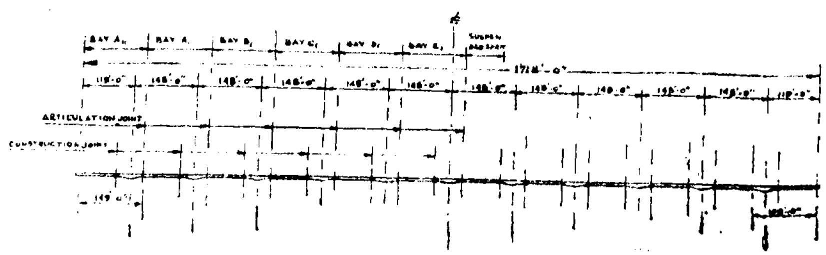

Fig. 1 (b). **During construction**

Thus, the deck structure can be divided into two different structural systems. one consisting of 6 girders and the other consisting of 5 girders joined by a suspended girder 88 ft long.

2.1.1. The clear roadway of the bridge is 22 ft with 3 ft wide footpaths on either side. The wearing course is of uniform thickness of $2\frac{1}{2}$ in. and is built in plain concrete with a specified strength of W_{28} Cylinder $= 4{,}500$ p. s. i.

2.1.2. To cut down the cost of earthwork in the approaches, the 149 ft long girders at both ends are given a longitudinal gradient of 1 in 74.5.

2.1.3. All the beams are prestressed with $1\frac{1}{8}$ in. diameter Macalloy bars in varying numbers and are connected together by R.C.C. cast-in-situ diaphragms spaced about 20 ft apart. The number of diaphragms has been determined in accordance with Morrice's distribution method.

Each 148 ft long girder was weighing about 77 tons. This was considered too heavy to handle with the available equipment at the site. The beam was, therefore, cast in two sections described as "span beam" and "support beam" having lengths of 88 ft and 60 ft respectively. One span beam weighs about 35 tons and the support beam about 31 tons. The precast units are cast on the river-bed on a specially prepared casting yard, at the position of the opening which the beams are meant to cover and are hoisted up after initial prestressing, by means of crab and winch and placed in position by a travelling trolley. Details of span and support beams are shown in Plates II & III. Both span and support beams are of I-section with a small top flange to act as castellations for supporting the R.C.C. cast-in-situ deck slab. The support beams have a depth of 9 feet at the support and the span beams 6 ft 10 in. at the centre. The width of the bottom flange is 24 in. and that of top flang 15 in. The thickness of the web has been determined by the formula $t = \frac{d}{40} + 4$ in., where d is the depth of the beam in inches. In this case, considering d as 6 ft 10 in., the web thickness of span beams provided is 6 in. At the hinges, the width of web of the upper tip as well as that of lower tip has been made 17 in. and it has been reduced to 6 in. after the end block in a length of $8\frac{1}{2}$ in. According to the practice followed in Lee-McCall system, the widening of the rib is made at 1 : 1 slope whereas in other systems, this slope is usually 4 lengths: 1 width.

2.1.4. The profile of the beam has been so designed as to allow unit construction of the form works for all the spans with slight modifications wherever necessary.

2.1.5. The prestressing cable comprises $1\frac{1}{8}$ in. diameter Macalloy bars. As soon as the concrete has attained a strength of 80 per cent of the ultimate designed strength, some of the rods were stressed by means of Lee-McCall jacks to take care of handling stresses. Precast units were then hoisted up in position, one end of the span beam resting on the abutment or the articulation as the case may be and the other end on a temporary pier, Photo 1, built for the purpose of supporting the ends of the span and support beams so long as they were not joined together. Thus, one end of the support beam also rested on the temporary pier and the other end cantilevered out from the pier to form articulated support for the next span beam.

2.1.6. The decking consists of $6\frac{1}{2}$ in thick R.C.C. slab continuous over 3 girders and cantilevering out at both ends to support the foot walks.

3. DESIGN STRESSES

3.1. The following stresses were adopted in the design:

(a) Allowable stress for ordinary R.C.C $f_c = 1,000$ p.s.i.

(b) Allowable stress for ordinary mild steel $f_s = 18,000$ p.s.i.

(c) Allowable stresses in Prestressed Concrete Works:

During service—

(i) Maximum compression in concrete—2,200 p.s.i.

(ii) Tension in concrete—nil.

(iii) Principal stress in concrete—$\frac{1}{30}$ of designed ultimate compression = 220 p.s.i.

(iv) Maximum tension in H.T. rod—after losses (16 per cent) = 35 tons per sq. in.

During construction—

(i) Maximum compression in concrete—2,500 p.s.i.

(ii) Maximum tension in concrete—$\frac{1}{10}$ of maximum compression, i.e., 250 p.s.i.

(iii) Maximum tension in high tensile steel rod—upto 0.1 per cent proof stress, which is about 50 tons per sq. in.

(d) Bearing pressures—

(i) Under Macalloy end plate—3,360 p.s.i. (local)

(ii) Under bridge bearings—1,250 p.s.i.

4. DESIGN CONSIDERATIONS

4.1. The bridge has been designed to meet the requirements of I.R.C. Bridge Code (Sections I to III), Draft Criteria for Prestressed Concrete Bridges and Indian Standard Codes for Prestressed Concrete. Since the designs were started prior to the publications of Codes, minor variations may appear.

4.1.1. The design has been prepared on the basis of the following assumptions and loading conditions:

(a) One unit of Class A loading in each lane moving abreast with footpaths fully loaded. (According to the present I.R.C. Code this condition of 2 lanes of load moving abreast may not be investigated.)

(b) Two units of Class A loading in each lane so spaced as to produce worst effects, the vehicles in each lane moving in opposite direction. No crowd load on footpath.

(c) One unit of Class AA loading.

(d) Load distribution factor for outer beams due to eccentric loading worked out in accordance with P.B. Morrice's method and found to be:

(i) For Class A loading—1.2 times mean load, corresponding to 1.17 of Courbon.

(ii) For Class AA loading—1.4 times mean load corresponding to 1.44 of Courbon.

(e) The full footpath live load assumed to be carried on the outer beams.

(f) The total dead load assumed to be shared equally.

(g) The effects of transverse load distribution as stated in (d) above are virtually eliminated for a simple support and do not appear in the unloaded span or in spans of a continuous structure. As such, the above transverse distribution factors were not taken into account while considering the effect of a span when load was placed on the cantilever or in any adjacent span.

(h) The effect of Poisson ratio neglected.

(i) The system of deck structure adopted here is a statically determinate one as the continuity has been broken by the insertion of a hinge in each span other than the end one.

4.2. Deck Slabs

4.2.1. The deck slab has been designed as continuous over the supporting beams with cantilevered footpath slabs on either side. The effect of haunches, though built monolithic with the slab, has been neglected and the slab has been assumed to be of uniform thickness for the purpose of calculating the stiffness and distribution factor. The effects of live loads for two types of Class AA loading have been taken into account and it is found that Class AA tracked vehicle governs the design.

4.2.2. The summary of moments is given in Table 1. Though moments have been given for faces of outer and inner supports and mid-sections, those at the beginning of haunches where

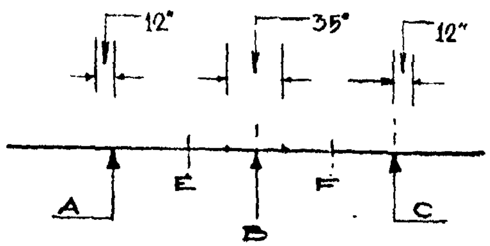

Fig. 2.

the depth of slab starts increasing, have also been worked out and sections checked both for concrete and steel stresses. Design moments are given in Table 2.

TABLE 1

Summary of moments

Loading			Bending moment in in.-lb		
			Critical sec. support A & C	Mid span	Critical section support B
Dead load			− 34,640	− 4,990	+ 8,870
LIVE LOAD	Footpath loaded	One side only	− 12,620	+ 1,815 − 5,445	+ 3,340
		Both sides	− 12,620	− 3,630	+ 5,500
	Class A on cantilever		− 67,600	−	−
	Class AA central over mid-support		−	+ 39,300	− 54,500
	Class AA one track at mid-span		−	+ 47,800	− 55,600

TABLE 2

Design moments

Section	Design moments in in.-lb	
	Positive	Negative
A or C	−	− 1,14,860
E or F	+ 44,625	− 10,435
B	+ 14,370	− 46,730
At the end of haunch of cantilever	−	− 66,000

4.3. Main Beams

4.3.1. The design of main beam is based on two considerations. Firstly, the precast beam section has to carry its own weight plus the weight of diaphragms, deck slab and the form works for the diaphragms and deck slab, and, secondly, the composite beam has to carry its own weight plus the weight of the wearing course, railings, etc., and the live loads. Fig. 3 (a) & (b) illustrates the two sections separately.

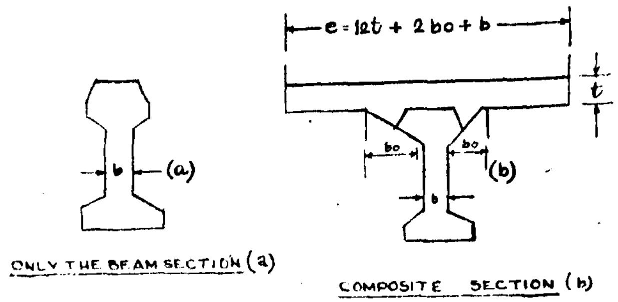

Fig. 3.

4.3.2. The structural considerations for the purpose of design are different for different bays A_1, B_1, etc. This is due to the fact that the reactions and moments from bay E_1 are transferred to the bays A_1, B_1, etc. For a typical bay, say E_1, the distributions of reactions at different stages of construction are illustrated in Fig. 4.

4.3.3. Similar dead load reactions calculated for all other bays are given in Table 3.

TABLE 3

Dead load reactions for the completed bridge

Bay	Left support in.-lb	Right support in.-lb
E_1	98,165	3,44,788
D_1	98,310	3,43,978
C_1	98,300	3,44,078
B_1	98,306	3,44,048
A_1	1,05,864	3,45,132

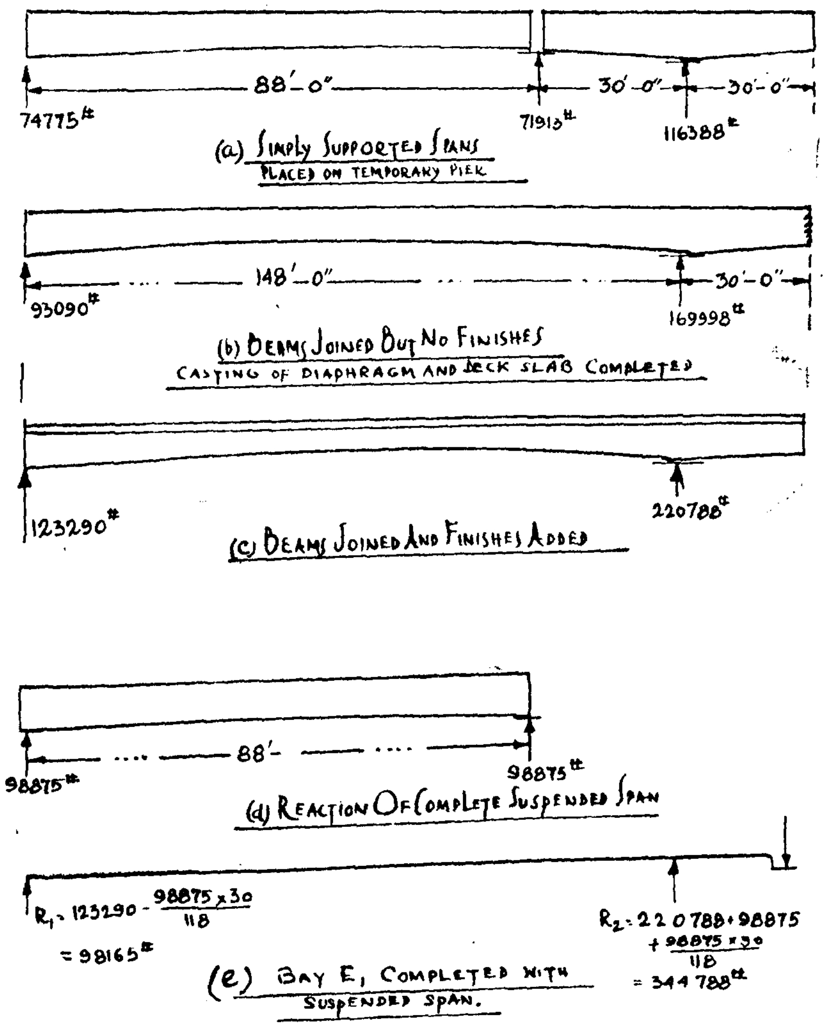

Fig. 4. Reaction in a bay E at different stages of construction

4.3.4. On similar considerations, dead load moments for different bays worked out are given in Table 4.

TABLE 4

Dead load moments at different stages expressed in in.-lbs × 10^6

Bay	Loading	Mid span	Critical section 41 ft from pier c/c	Constn. joint	Pier C_L
Similar for each bay (Typical)	Beams + decks + diaphragms before jointing	17.95	7.85	0	− 10.89
	Beams + decks + diaphragms after jointing	27.05	24.84	19.31	− 10.89
	Increase in moment due to jointing	9.70	16.99	19.31	0
	Own footpaths + hand-rails + finishes	9.62	8.52	6.49	−2.95
	Reaction from the suspended span — R_1 = 25,125 lb				
E_1	Due to reaction from the suspended span	−13.24	−23.20	−26.50	−35.60
	Own footpaths + hand-rails + finishes	−3.62	−14.68	−20.01	−38.55
	Reactions from the bay $E_1 + S$ — 0.254 × 98,165 = 24,980 lb				
D_1	Due to reaction for $E + S$	−13.15	−23.1	−26.34	−35.3
	Due to footpaths + hand-rails, etc.	−3.53	−14.58	−19.81	−38.25
C_1	Due to reaction from D_1 & $E_1 + S$	−13.2	−23.1	−26.4	−35.4
		−3.52	−14.58	−19.91	−38.35
B_1	— Same as bay C_1				
A_1	— Same as bay C_1				

4.3.5. Since the span of the end girder is different from that of other spans, it requires special treatment. The dead load moments at different stages worked out are given in Table 5.

TABLE 5

Dead load moments for end spans at different stages expressed in in.-lb × 10^6

Bay	Loading	Mid span	Critical section	Constn. joint	C_L of pier
A_{11}	Self weight of beam + deck + diaphragms before jointing	18.4	7.92	0	−10.89
	Self weight + deck + diaph. after jointing	28.40	25.20	19.6	−10.89
	Increase in moment after jointing	10.00	17.28	19.6	0
	Due to own footpaths, etc. + other base completed	−3.63	−14.55	−19.9	−38.35

4.3.6. Next, live load moments at different sections are calculated. The values of design live load moments under worst condition of loadings for end bays are given in Table 6. In this connection, it may be mentioned that loads to the right of span

TABLE 6

Live load moments for end spans at different sections expressed in in.-lb × 10^6

Name of beam	Moment	Load condition		45 ft from abutment end	41 ft from pier end	Constn. joint	Pier	Remarks
Outer beam	Normal	1		30.94	29.70	25.06	36.60	
		2	a	28.73	29.40	25.75	33.60	
			b	30.45	31.05	27.13	35.70	
	Reverse	1		11.49	20.14	22.73	7.92	
		2	a	10.15	17.70	20.10	7.10	
			b	10.95	19.10	21.67	7.60	
Innner beam	Normal	1		22.80	22.00	18.50	27.10	
		2		24.33	25.15	22.15	28.00	
	Reverse	1		9.9	17.35	19.60	6.85	
		2		10.15	17.70	20.10	7.10	

under consideration will cause moments in the span. For this reason, there will be reversal of moments when alternate spans are loaded. In the Table, "normal" means positive moment in the section and "reverse" means negative moment in the section. The positions of live loads for worst effect, have been determined by drawing "Influence Lines".

Note – Load condition 1 : 2 lanes at Class A loading heading in the same direction with crowd load and no other load on the bridge.

Load condition 2 (a) : 2 lanes of Class A loding heading in opposite direction in the span plus similar loads placed in the alternate spans, but without crowd load in any span.

Load condition 2 (b) : Similar to load condition 2 (a) but with 50 per cent crowd load in the span.

4.3.7. The high tensile bars are stressed to 45 tons-per sq. in. with Lee McCall Jack, allowing for friction losses and jack losses. Six bars and 2 bars are initially stressed respectively for span and support beams in order to take care of handling stresses.

4.3.8. Stresses at various stages of construction are given in Table 7 for bay A_{11}.

TABLE 7 (a)

Stresses at mid-section of bay A_{11}

Stage	Outer beam		Centre beam	
	Upper fibre	Lower fibre	Upper fibre	Lower fibre
(a) Dead load of 88 ft shore span beam	+ 773	– 594	+ 773	– 594
(b) Due to initial prestress (without loss)—				
(i) 6 bars 1, 2, 3, 6, 7 and 8 for outer beam	– 619	+ 1,759	—	—
(ii) 5 bars 1, 2, 3, 7 and 8 for centre beam	—	—	– 551	+ 1,497
(c) Due to (b) but after losses (16 per cent)	– 520	+ 1,478	– 463	+ 1,258
(d) Due to (a) and (b)	+ 154	+ 1,165	+ 222	+ 903
(e) Due to (a) and (c)	+ 253	+ 884	+ 330	+ 664

TABLE 7 (a) (*Contd.*)

Stage	Outer beam		Centre beam	
	Upper fibre	Lower fibre	Upper fibre	Lower fibre
(f) Due to self weight of the beam + deck + diaphragm before jointing	+1,495	– 1,160	+ 1,495	– 1,160
(g) Increase in stress due to jointing	+ 199	– 426	+ 199	– 426
(h) Due to second stage prestressing of 6 Nos. $1\frac{1}{8}$ in. and 3 Nos. $1\frac{1}{8}$ in. dia. (after losses 16 per cent)	+ 208	+ 1,242	+ 208	+ 1,242
(i) Due to footpaths, finishes and other bays	– 70	+ 134	– 70	+ 134
(j) Due to effective prestress after the bridge is complete but without live load (a)+(f)+(g)+(h)+(i)	+ 1312	+ 1,268	+ 1,369	+ 1048
(k) Due to live load				
(a) Normal moment	+ 598	– 1,146	+ 470	– 901
(b) Reverse moment	– 222	+ 426	– 196	+ 376
(a)	+ 1,910	+ 122	+ 1,839	+ 147
(b)	+ 1,090	+ 1,694	+ 1,173	+ 1,424

TABLE 7 (b)

Stresses at the critical section

(a) Beams + deck + diaphragms before jointing	+ 558	– 426	+ 558	– 426
(b) First prestressing after losses	+ 50	+ 986	– 81	+ 915
(c) Increase after jointing	+ 315	– 647	+ 315	– 647
(d) Second prestressing after losses	+ 429	+ 895	+ 429	+ 895
(e) Footpaths, finishes and other bays	– 256	+ 500	– 256	+ 500
(f) Bridge complete	+1,096	+1,308	+ 967	+1,237
(g) Due to live load — Normal moment	+ 524	–1,025	+ 443	– 866
(g) Due to live load — Reverse moment	– 355	+ 695	– 312	+ 610
(h) Service — Normal	+1,620	+ 283	+1,498	+ 371
(h) Service — Reverse	+ 741	+2,003	+ 653	+1,847

TABLE 7 (c)

Stresses at the construction joint

Stage	Outer beam		Centre beam	
	Upper fibre	Lower fibre	Upper fibre	Lower fibre
(a) Beams + deck + diaphragms after jointing	+ 306	− 485	+ 306	− 485
(b) Prestress after losses	+ 425	+ 676	+ 438	+ 564
(c) Finishes, footpaths and other bays completed	− 299	+ 478	− 299	+ 478
(d) Bridge complete	+ 432	+ 669	+ 445	+ 557
(e) Live load moments: Normal	+ 387	− 619	+ 333	− 532
Reverse	− 342	+ 546	− 302	+ 483
(f) Service: Normal	+ 819	+ 50	+ 778	+ 25
Reverse	+ 90	+1,215	+ 143	+1,040

TABLE 7 (d)

Stresses at pier support

Stage	Outer beam		Centre beam	
	Upper fibre	Lower fibre	Upper fibre	Lower fibre
(a) Beam + deck + diaphragms	− 500	+ 358	− 500	+ 358
(b) First prestressing	+ 511	− 129	+ 511	− 129
(c) Footpaths, finishes and other bays	− 522	+ 862	− 522	+ 869
(d) Second prestressing	+ 1,129	− 345	+ 1,033	+ 314
(e) Bridge complete	+ 616	+ 753	+ 522	+ 784
(f) Live load: (a) Normal	− 498	+ 830	− 382	+ 635
(b) Reverse	+ 108	− 179	+ 97	− 161
(g) Service: (a) Normal	+ 120	+ 1,583	+ 140	+ 1,419
(b) Reverse	+ 726	+ 574	+ 619	+ 623

4.3.9. Stresses at the articulation have been checked for conditions, namely, (i) only the next bay has beeen completed, (ii) all the bays have been completed.

The stresses are found as given below:

Condition (i) – $ft = +149$ p.s.i.
$fb = +489$ p.s.i.

Condition (ii) – $ft = +151$ p.s.i.
$fb = +392$ p.s.i.

Maximum shear stress occurs under condition (i) and works out to 5 p.s.i.

4.3.10. The values of maximum shear forces and reactions are given below for end bay, A_{11}, only. The calculations have been made with the help of influence lines.

TABLE 8 (a)

Maximum shear force

Section	Loading condition	Outer beam	Centre beam
Support section	I	288,354 lb	257,074 lb
Abutment section	I	198,130 lb	177,320 lb

TABLE 8 (b)

Maximum reaction

Section	Loading condition	Outer beam	Centre beam
Support section	I	483,450 lb	448,130 lb
Abutment section	I	198,130 lb	177,320 lb

4.3.11. Shear Stress

The shear stresses at different sections have been calculated by the formula—

$$q = \frac{V}{Ib}\int_{y_1}^{h} bydy$$

which reduces to

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

Where q is the shear stress, V the shear force, I the moment of intertia, b the width of the section considered, and g the moment of the area of all elements lying above the plane considered. The principal stresses have been calculated with the help of the formula

$$p = \frac{f}{2} \pm \sqrt{\frac{^2f}{4} + q^2}$$

which is, for principal tensile stresses,

$$P_t = \tfrac{1}{2}(\sqrt{f^2 + 4q^2} - f)$$

where f is the bending stress and q the shear stress. At the extreme fibre f is maximum but $q = O$ and at netural axis, $f = O$, but q is the maximum. Hence, for a section lying between the extreme fibre and the neutral axis, the principal tensile stress will be the maximum. Usually such a section may occur at either of the three sections, viz., the junction of the top portion of the web with top slab, the neutral axis and the junction of the bottom portion of the web with the bottom flange of the girder. Stresses at these three sections have been checked. The values of principal tensile stresses, as found out, for abutment support and pier support sections are 135 and 222 p.s.i. respectively. In calculating the shear force at any section, effect of prestressing force and haunching in reducing the shear forces, has been taken into account. Formulae used are P Sin θ and C tan α, where P is the prestressing force, θ, the inclination of the tendon, and C, the total compressive force and α, the angle which the soffit slab makes with the horizontal and is considered positive in an anti-clockwise direction starting from the haunch.

Shear reinforcements have been calculated according to Guyon's theory. The general formula used is

$$X = \frac{fZAsfs}{2q\,V}$$

Where X is the spacing in inches, f the compressive stresss in p.s.i., A_s the area of shear steel in sq. in., fs the elastic limit of mild steel (taken equal to 30,000 p.s.i.) q the shear stress in p.s.i, V the total shear in lb after considering the effect of prestressing force and haunches and $Z = \frac{I}{H}$ in inches.

$$[H = \Sigma(A \times y)]$$

In some of the cases, q has become greater than $f/2$ for which the formula has been modified as

$$X = \frac{Z\,As\,fs}{V\,tan\,45^\circ}$$

4.4. Cross Beams

4.4.1. The cross beams are designed as ordinary R.C.C. tee-beam having depth of beam as 79 inches, top flange width as 84 inches and the thickness of web as 6 inches. The moments due to live load have been worked out on the basis of the method evolved by Morice and Little, and described in detail in "Structural Engineer" Vol. 32, No. 3. March '54, pp. 83-111. The maximum moment is caused by Class AA loading and works out to 2,105 in. kips. The reinforcements required are 6 numbers 3/4 in. diameter bars.

4.5. End Blocks

4.5.1. The end blocks have been designed in accordance with the method outlined in Magnel's book on Prestressed Concrete.

4.6. Bearings

4.6.1. Cast steel roller and rocker bearings have been used for all spans including suspended span. Rocker bearings are provided over all pries and at one end of the suspended span. Roller bearings are provided over abutments, at all articulation points and at one end of the suspended span. The design has been based on formula given in Scott's book on "Reinforced Concrete Bridges". The value of k has been taken as 2840.

All bearings are provided with dust proof cover.

4.7. Ultimate Strength

4.7.1. Though the design has been made on the basis of "Elastic Theory", its strength at ultimate load has been calculated and has been found to satisfy the following criteria:

For steel resistance—$M_{ult.} = 1.5$ (D.L.M.)
+ 4.226 (L.L.M.)
or 2.68 (D.L.M. + L.L.M.)

For concrete resistance—$M_{ult.} = 2.83$ (D.L.M + L.L.M.)

5. MATERIALS

5.1. Aggregate

Two different types of aggregates were tested in the West Bengal Road and Building Research Laboratory and after tests, it has been decided to use Pakur chips which are of trap variety. Field tests were carried out with different gradings of coarse aggregate and the results are found to be satisfactory if Pakur chips in two grades 3/4 in. to 3/8 in. and 3/8 in. to 3/16 in. are mixed together in definite proportions. The combined fineness modulus obtained is 6.5. Sand available in the river bed has been tested in the laboratory and is found to be suitable for concrete works. The fineness modulus obtained is 2.64.

5.2. Mild Steel

Steel complying with I.S. specification was used.

5.3. Prestressing Bars

Macalloy bars complying with I.S. specification was used.

5.4. Concrete Mix

Several tests were carried out with a 200-ton testing machine installed at the site on samples of concrete blocks prepared by mixing the above materials, mixed in different proportions by weight for prestressed concrete works. Based on 31 sets of tests carried out with different brands of cement and different proportions of coarse and fine aggregates, it was decided to use the following mix by weight which gave satisfactory results-

Grade B 6,600—Coarse aggreaate (dry): $\left.\begin{matrix}\text{3/4 in. to 2.36}\\ \text{3/8 in. to 0.64}\end{matrix}\right\}$ 3.00

Sand (dry).................................1.5

Cement..............................1

W/C ratio..........................0.40

The mix works out to be (1 : 1.5 : 3) by weight and 1 : 1.41 : 2.8 (by volume).

Yield of concrete was found to be 4.10 cu. ft. per bag of cement and the number of bags required per 100 cu. ft. of concrete was 24.5 bags. The average cube strength obtained during tests was 9,000 lb per sq. in.

6. SPECIFICATIONS

6.1. Concrete in Superstructure

(a) Precast girders and construction joints—Grade C 6,600 —strength 6600 p.s.i.

(b) Cast-*in-situ* diaphragms and cross beams—Grade C 3,750—strength 3,750 p.s.i.

(c) Deck slab excepting for a stretch of 2 ft behind the end anchorages—

Grade C 3,750—*strength* 3,750 p.s.i.

(d) 2 feet stretch behind the anchorage in deck slab—Grade C 5,125—strength 5,125 p.s.i.

(e) Wheel guards and footpaths—Grade C 3,750—strength 3,750 p.s.i.

(f) Railings, posts, etc—$1 : 1\frac{1}{2} : 3$

(g) Wearing course—$W/_{28}$ = 4,500 p.si.

6.2. Grout Mix

6.2.1. Compressive strength of cubes at 7 days should not be less than 3,000 lb per sq. in. and water-cement ratio should not be more than 0.5. The shrinkage of neat cement paste should not be more than 2 per cent by volume. If the shrinkage is more, sand is to be added. Actually the shrinkage of neat cement was found to be within permissible limit and hence neat cement paste was used.

6.3. Macalloy Bars

6.3.1. Ultimate tensile strength – 64 – 72 tons/per sq. in.

Secant modulus at 40 tons per sq. in – 25×10^6 lb per sq. in.

0.1 per cent proof stress – 50 tons per sq. in.

0.2 per cent proof stress – 58 tons per sq. in.

Elongation on 8 in. gauge length – 6 per cent minimum.

6.3.2. Samples of H.T. rods received from supplies at site were tested at random at the Government Test House, Alipore, and were found to comply with the above specifications. No cold bend test was made as it was not specified in the tender.

7. CASTING YARD

7.1.1. Ascertaining what equipment was readily available the designer planned the casting and erection of the beams in such a way that a crab and winch and travelling trolley would suffice. The advantage of dry river bed in the working season was also utilised. The weight of each individual unit was kept very low, i.e., within 40 tons, so that two 20-ton travelling chain pulley blocks or ordinary crab and winch with two 20-ton pulley blocks would be able to hoist beams in position.

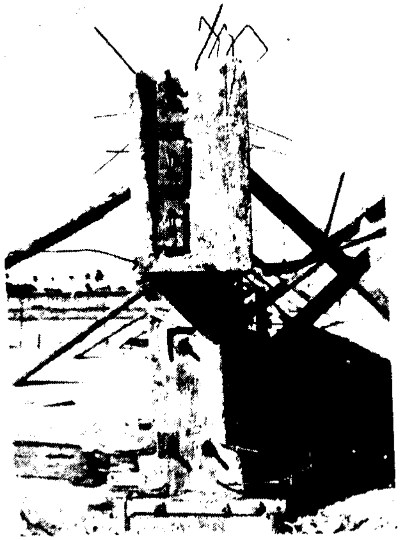

Photo 5

End plates in position at articulation point

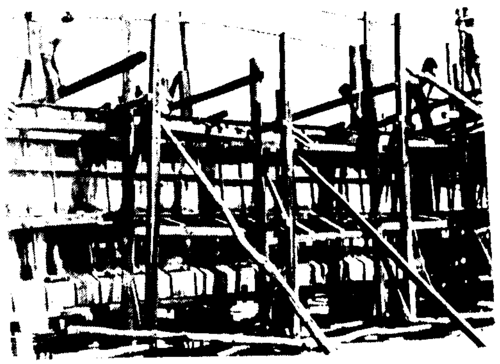

Photo 6

Inspection windows may be seen on the top of bottom flange

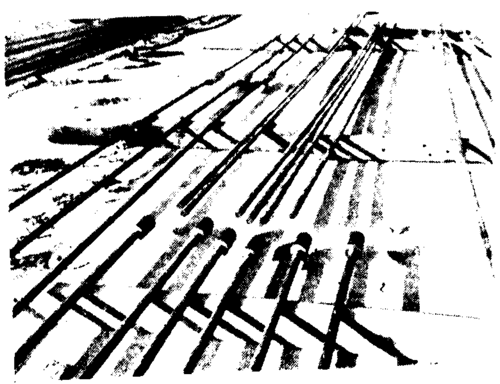

Photo 7

Couplers of Macalloy bars

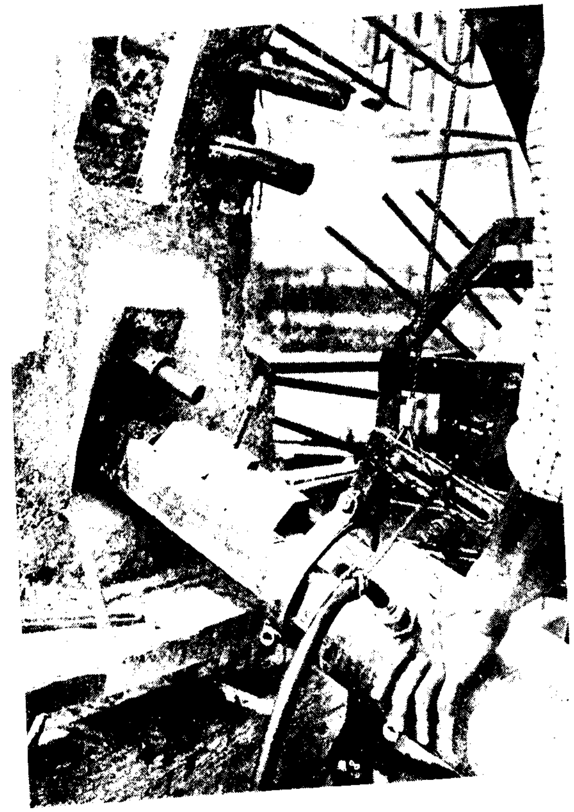

Photo 8

Jack in action

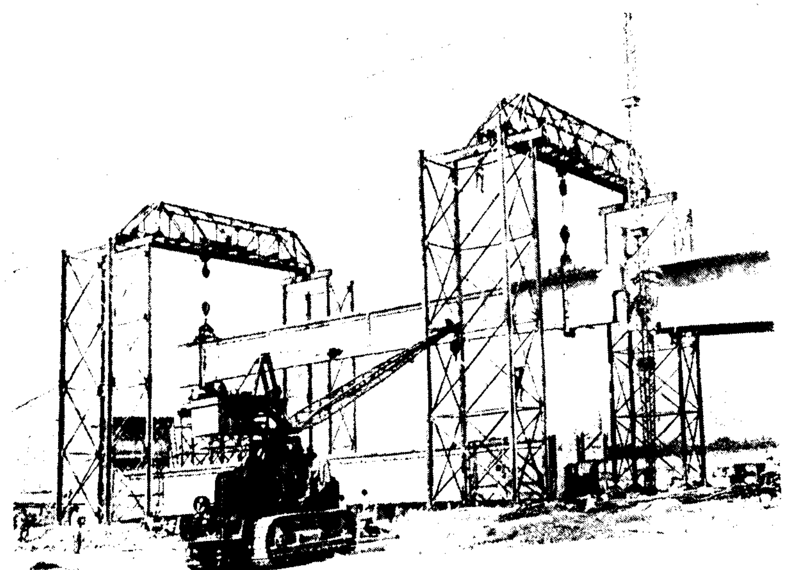

Photo 9

Hoisting of the beam

7.1.2. As the travelling trolley mounted on gantry girders could move only across and not along the length of the bridge, the fact was taken into consideration while casting the beams on the river bed. Casting beds for span beams and support beams were staggered in position so as to facilitate initial prestressing.

8. Casting Bed

8.1. The casting bed was made of inverted concrete T-blocks spaced 4 ft centres at each end, supported on 12 ft × 4 ft sleeper cribs laid on well compacted sandy bed. These sleeper cribs at the ends distributed the load on a greater area when after initial prestressing the beam was self-carrying. A series of similar blocks were placed at 4 ft centres to provide intermediate supports to the form work. The T-blocks were one foot six in. wide, three feet long and of varying depths. They were designed in such a way that the load of the beam on each block (about $1\frac{1}{3}$ tons) was distributed on the sandy bed over an area of 4.5 sq. ft., thus bringing down the bearing pressure to 1/3 ton per sq. ft.

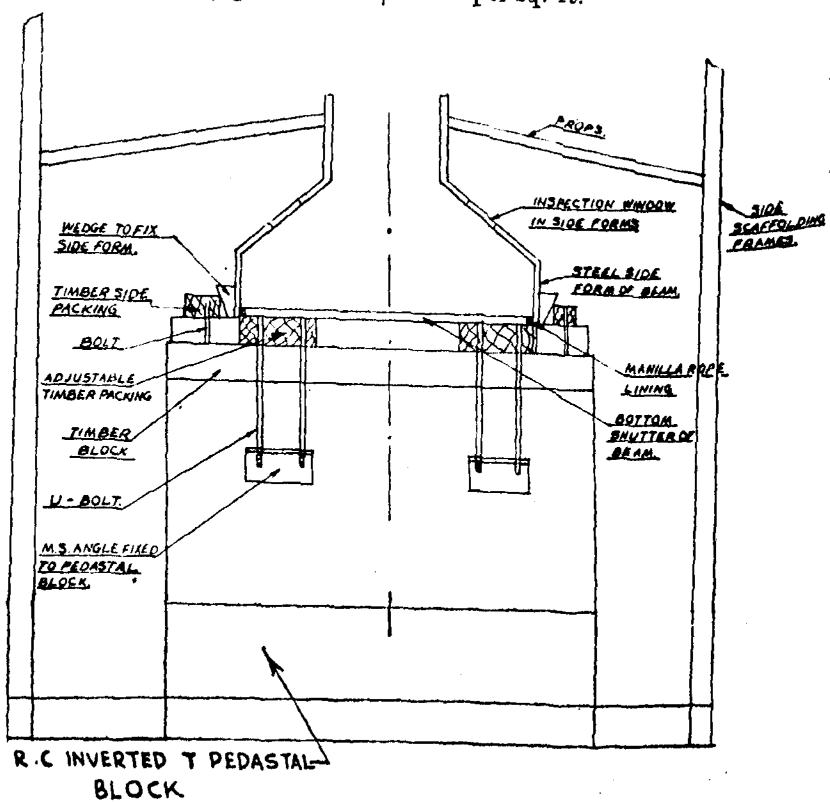

Fig. 5.

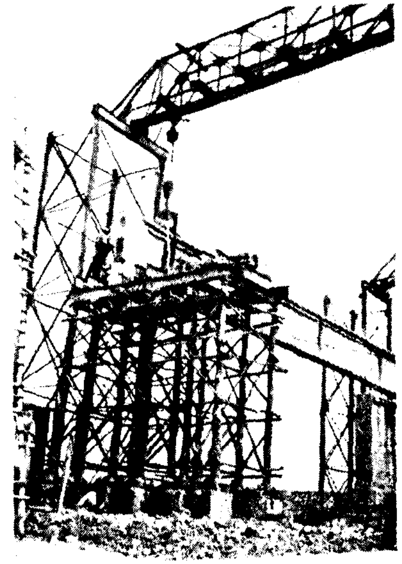

Photo 1
Temporary pier of salballahs

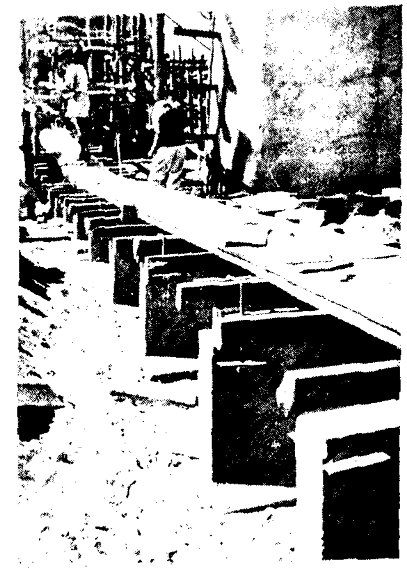

Photo 2
General arrangement of casting bed with soffit shutter in position

Photo 3
H. T. rods in position

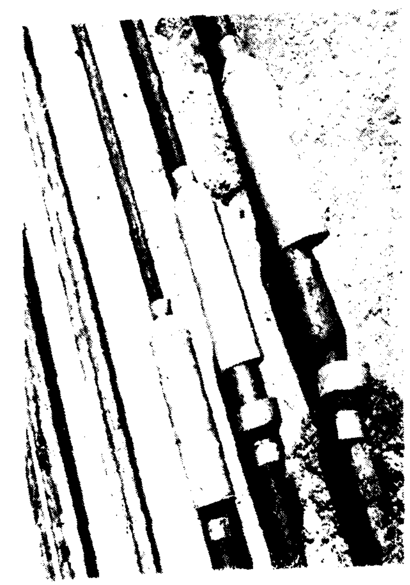

Photo 4
Locally made sheathing for coupler

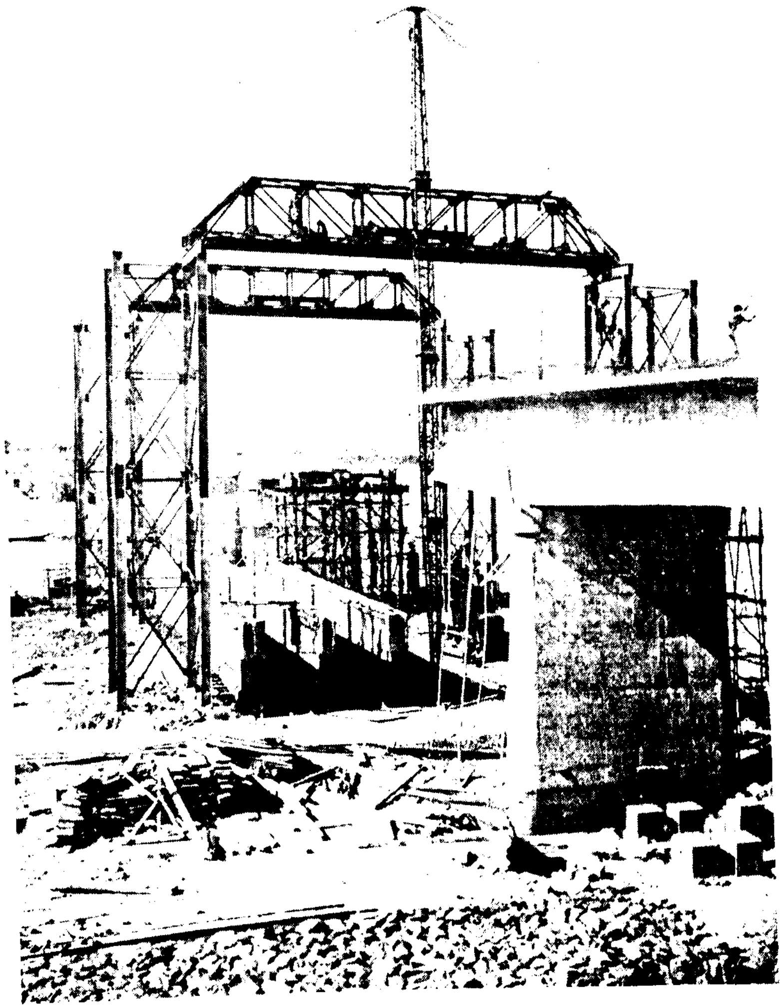

Photo 10

Tower in action

Over these concrete blocks a timber cross piece 3 feet long by 3 inches wide and 5 inches deep was placed. Adjustible packings were necessary to fix the bottom shuttering of the beam at correct level. Rubber packing pieces were inserted between the bottom shuttering and the packing pieces to absorb shocks due to vibrations caused by concrete vibrators and to prevent the same from being transferred to the blocks, Photo 2.

9. FORM WORK FOR BEAMS

9.1. The bottom shuttering was of $1\frac{1}{2}$ in. thick dressed wooden plankings connected together by tongue and groove joints and stiffened at intervals by wooden battens 4 in. by 3 in. In order to prevent leakage of mortar during casting and also to obtain a smooth finished surface, the upper surface of the planks was lined with black sheets. Side shutters were made of $\frac{1}{8}$ in. thick steel plate. They consisted of 3 ft by 4 ft units stiffened with $1\frac{1}{2}$ in. $\times$ $1\frac{1}{2}$ in. $\times$ $\frac{1}{4}$ in. M.S. angles and $1\frac{1}{2}$ in. $\times$ $\frac{1}{4}$ in. M.S. flats at centres and ends with bolt holes for jointing the separate pieces. Apart from fixing the bottom shutters with side shutter by means of wedges fixed to side battens, the junctions were lined with manila ropes to prevent leakage of mortar. This arrangement worked out very satisfactorily and practically no leakage of mortar was noticed.

9.2. Arrangement for fixing form vibrators to the bottom and side shutters was made in such a way that a vibrator would vibrate about 9 square feet of area. Inspection windows were also kept about 4 feet apart to see whether concrete flows to all places.

9.3. Scaffolding frames of 4 in. $\times$ 3 in. timber battens braced with 3 in. $\times$ 2 in. planks were erected on both sides to support the working platform as well as the props for the side forms.

10. PROCEDURE FOR CASTING OF THE BEAMS

10.1. Placement of Reinforcement

Before placement of reinforcement, the transverse level of the bottom shutter was checked with a spirit level and the longitudinal layout with a theodolite. It is very important that the alignment of the girder should be set accurately, as otherwise the jointing of the two sections involving coupling of H.T. rods would be difficult.

Mild steel reinforcements were next laid and supported temporarily on battens fixed with the scaffolding frames. Side shutter was then erected on one side Photo 3.

The Macalloy bars with thread type anchorages were imported in exact lengths with threads on both ends. Any damage to screw threads would lead to rejection of the whole piece, as the special die required for cutting screw threads was not usually imported along with the bars. However, to cover up probable losses during transit or handling, about 2 per cent extra pieces with necessary couplers were also imported. This drawback of Macalloy bars, it is understood, is being overcome by the introduction of cone type anchorages.

The threads of the bars and couplers were cleaned with oil and wire brush after removing thread masks. Bars were then coupled where necessary in each beam (maximum rolling length 60 ft) and were inserted in the lead coated sheaths, known as Kopex tubings which were imported. Special sheaths for couplers, which were larger in size than the H.T. rods, were necessary to ensure free movement. The joints in the Kopex tubings were made by solderings and then covered with black tape. Due to import restrictions, the use of Kopex tubing was dispensed with in favour of sheaths locally manufactured with 24 gauge plain G.I. sheets, Photo 4. These locally made sheaths were found to be quite satisfactory, but these were less flexible than the Kopex tubings. For this reason, the locally made sheaths could not be used at sharp bends.

The Macalloy bars were then placed in position and were supported at intervals on transverse rods which were welded to stirrups. The profiles of the bars were checked by measuring depths from a level surface at top and comparing with the theoretical profiles furnished by the design office. The other side shutter was then fixed in position. Extra ½ in. dia. M.S. rods were inserted for additional temporary supports for the H.T. bars which were removed when the concrete started setting. The holes left were grouted with rich cement mortar.

10.2. Fixing of End Plates

The anchorage units of Lee McCall system are very simple and consist of an end plate with high efficiency nuts. The end plates are usually 1½ in. thick. Due care had to be taken to set them correctly in position at desired inclinations. For this purpose, prefabricated timber shutters with necessary packing pieces were used for ensuring this inclination.

For facilitating grouting, Lee McCall grouting flange was bolted to a specially slotted end plate as shown in Fig. 6. The grout is pumped through a ½ in. diameter hole in the end plate.

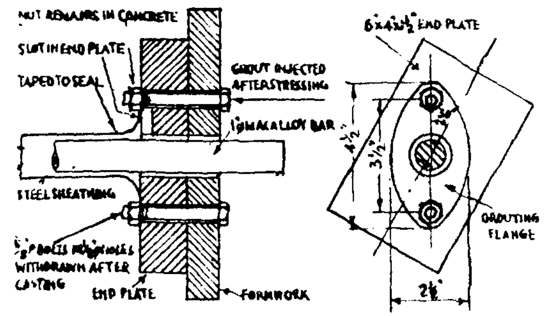

Fig. 6. (By courtesy of M/s. McCall Ltd., England)

Care must be taken during fixing of the grouting flange to see that it fits squarely with the end plate.

10.3. Checking of all reinforcements, profiles of H.T. bars, lapping of Kopex tubing, etc., and fixing the forms was completed one day in advance of the schedule date of casting. Routine checks were made frequently on the samples of coarse and fine aggregates for determination of moisture content, void ratio, gradation, uniformity, co-efficients, unit weight, specific gravity, silt content, etc. Since no variation was observed during this check, the standard mix was used throughout. One tilting and one non-tilting mixer machines of 7 cubic feet capacity each were used for mixing the concrete.

10.4. Batching of the concrete for all prestressed concrete units was done by weight.

The moisture contents in sand and coarse aggregates were determined from time to time and weights of the field mix were adjusted accordingly. The actual mix used in the construction was (1 : 1-1/3 : 2-2/3) by weight, which is slightly richer than the design mix. This rich mix was proposed to obtain the designed strength of concrete early so that the initial prestressing can be done after a week or so.

10.5. Slump tests were performed on the mix regularly. In the case of a stiff mix, proper compaction could only be assured by proper vibrations for which the following vibrators were used :

(1) Trillor brand electrically operated form vibrator for bottom and side forms—5 Nos.

(2) Holman brand pneumatic hammer vibrators for side forms operated by compressed air at a pressure of 80 p.s.i.—5 Nos.

(3) Ingersol brand pneumatic immersion vibrator fitted with 2 inches diameter needle operated by compressed air at a pressure of 80 p.s.i.—3 Nos.

(4) Baxtul brand internal vibrator with 2 in. needle operated by petrol engine—2 Nos.

(5) Master brand electrically operated internal vibrators —2 Nos.

(6) Special electrically operated internal vibrators with tapered small diameter needles—2 Nos.

The external vibrators were fixed to the bottom shutter at a distance of 4 ft. The side form vibrators were arranged in such a way that it would follow the line of flow of concrete. Usually, the spacing was 3 ft staggered on the both faces.

10.6. Though the concreting of the full depth of the beam in a continuous operation without permitting any construction joint was aimed at, it was observed during actual concreting that the flow line of the concrete poured from the top did not advance at the desired inclination of 30 to 45 degrees. Actually, the inclination tended to become vertical with the result that cavities were noticed at several places. For this reason, it was decided to cast the beam in two layers, top and bottom, separately in quick succession. The time lag between the two castings was limited to 2 hours.

10.7. To avoid excessive vibration causing segregation of aggregates, the external form vibrators were switched on only when the beam was partially filled with concrete. Internal vibrators were also very carefully used, avoiding hitting the sheaths of the H. T. bars.

10.8. The concreting was started from the end block and proceeded towards the construction joint. The total volume of the concrete in each span beam was about 500 cubic feet and the casting took about 10 hours.

10.9. Immediately after the concreting was completed, all the Macalloy bars were hammered in and out slightly to check that no mortar leaked into the ducts and clogged the H. T. bars. The same operation was also repeated next day.

10.10. For assessment of the strength of the concrete, six 6 in. cubes were taken from different batches at regular intervals.

11. REMOVAL OF FORMS

11.1. The side forms were removed within 24 hours after completion of casting. The surface of the beam was found to be generally good except at the joints of the shuttering pieces which were rubbed smooth later on with a carborundum block. Honey-combing was rare but when detected was repaired with new concrete.

12. PRESTRESSING

12.1. Equipment for Prestressing

The following equipment and accessories were used for the stressing operation :

(1) 2 Nos. 42 ton jacks No. II

(2) Adapter

(3) Cotter

(4) Packing plate for use when excessive number of washers was required

(5) Oil can containing hydraulic oil

(6) Adapter inserts

(7) Ring spanners

(8) Tommy bars

(9) Spare hose and gland washers.

12.2. Lee McCall Jack No. II.

The prestressing jack and the hydraulic pump are mounted on a trolley fitted with a winch for handling the jack and a tool box containing the accessories. Mark II jack which has a travel of 6 in. can be used for all units irrespective of their length. The working space rquired for members of lengths varying from 80 to 120 ft is 4 ft and 3 in. This space was provided while laying out the casting yard. The suspension cable and swivelling head of the rig of the trolley enables prestressing of the bars at any position. The trolley was placed on a 6 ft elevated wooden table and all the bars were stressed from one location.

12.3. In stressing operations, which are simple, the following staff was employed :

Labour for pumping — 2 Nos.

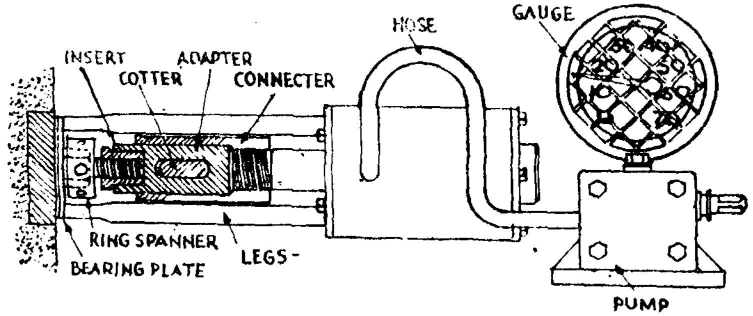

Fig. 7. (By courtesy of M/s. McCall Ltd., England.)

Jack man for reading gauge — 2 Nos.
Extension reader — 2 Junior Engineers
Nut tightener — 2 Mates
Supervisor — 1 Assistant Engineer

12.4. Hogging of the beam after prestressing was measured by ordinary levelling instrument. Actual deflection was found to tally, within fair range, with the theoretical value when the value of E_c was assumed between 5×10^6 to 6×10^6 p.s.i.

12.5. The general arrangement of the jack for-stressing is shown diagrammatically in Fig. 8. The requisite diameter of insert was screwed home into the adapter body and the adapter and the insert were temporarily screwed on to the extended thread of the bar. The jack mounted on the trolley was then connected by the cotter to the adapter and the thrust was transmitted through the legs of the jack to the end anchor plate of the unit. The jack was actuated by a hand operated hydraulic pump and for the facility of extension measurement a vernier scale was fitted to the jack. The jacking pressure was recorded by a pressure gauge mounted on the jack trolley.

12.6. Anchorage Components

The details of anchorage components of the Macalloy bars are shown in Fig. 9.

Macalloy bars are anchored to the end plate by means of special nuts bearing on washers. In a long member, it is necessary to couple two bars together to form one unit. For this purpose, a special coupler is used which is, in fact, two special nuts in one

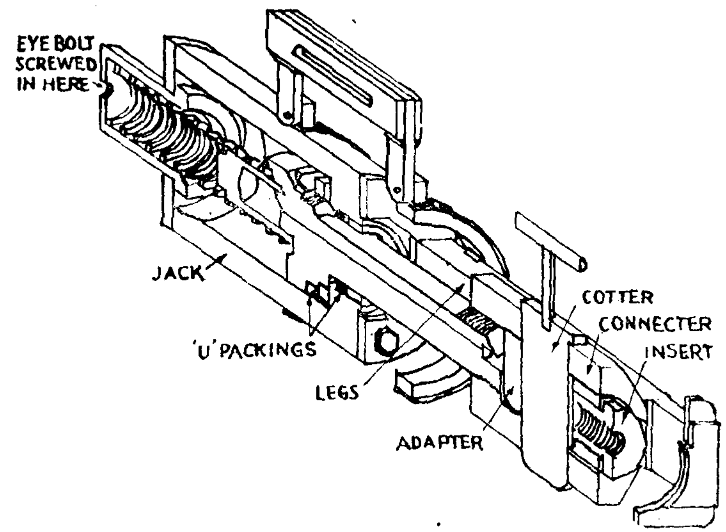

Fig. 8. (By courtesy of M/s. McCall Ltd., England.)

List of end Anchorage Components

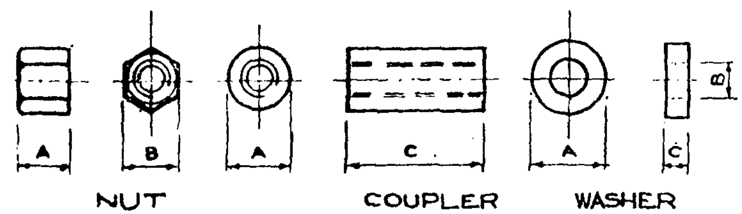

Fig. 9. (By courtesy of M/s. McCall Ltd., England.)

piece. The nut and the coupler are bell mouthed to obtain the high efficiency fit. Washers are of variable sizes depending on the size of the bar and are usually 3/16 in., 1/2 in. and 3/4 in. thick.

Threads of Macalloy bars are cut with special tools in such a way that a Lee McCall nut when run half-way up the thread will develop 85 per cent strength of the plain bar. If the nut is run up to the hand tight position and then tightened with a spanner so that threads in the bell-mouth of the nut are forced further up in a spanner tight position, a strength of nearly 100 per cent of that of the plain bar will be obtained. It was seen during actual stressing that only moderate spanner pressure was required to bring a nut from the hand-tight position to the spanner-tight position and the movement of the nut ranges from 1/16 in. to 1/8 in.

12.7: Sequence of Stressing

12.7.1. The stressing of the bars should be so phased that stresses at any stage of construction should not exceed the permissible limits. It was, therefore, necessary to follow a definite sequence. The first stressing was done for handling and placement of the beams over the temporary pier. The second stressing was done after jointing, casting of diaphragms and a portion of the deck slab. The third stressing was done after casting another portion of the deck slab and the final stressing after completing the remaining portion of the deck slab. The details are given below :

(a) Bars 1, 2, 3, 6, 7 and 8 of outer girders were stressed for handling when the cube strengths attained 80 per cent of the designed ultimate strength. Usually, the required strength was obtained after 8 to 10 days.

(b) Bars 1, 2, 3, 7 and 8 of inner beams were stressed for handling.

(c) Bars 21 and 22 of support beams were stressed for handling.

(d) Restressing of bars 1, 2, 3, 7 and 8 of the outer girders was done to pick up losses due to elastic shortening. This was done conveniently by tightening the nuts again. This method of making up losses due to elastic shortening is particularly suitable for Lee McCall system.

(e) Restressing of bars 1, 2, 3 and 7 of inner beams was done to pick up losses due to elastic shortening as in (d) above.

(f) Both span and support beams were erected and supported ;on temporary pier with a gap of 2 ft exactly.

(g) Bars across joints were coupled. Diaphragms and the gap between the ends of the girders over temporary pier were concreted. Difficulty was experienced in coupling the bars across the joint in the case of first beam due to defect in casting. Afterwards, no further difficulty was experienced.

(h) Deck slab concreted from A to B.

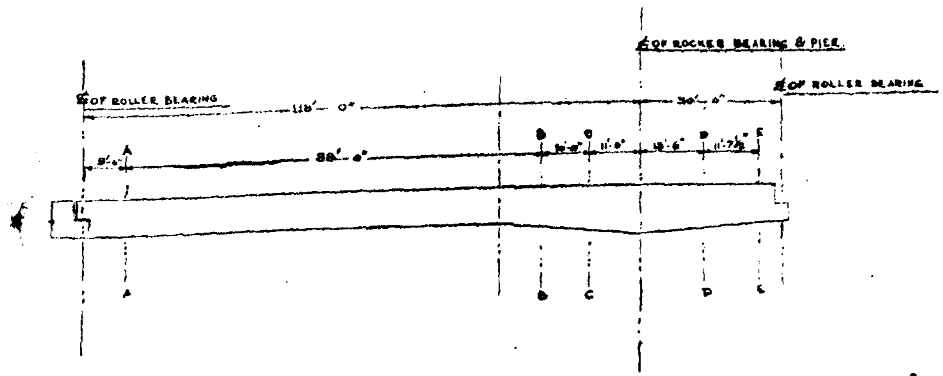

Fig. 10.

(i) Temporary fixing plates of bearings were removed to allow the rollers to come to their normal position after stressing. Initially, the rollers were not put in their normal position in consideration of elastic shortening due to stressing.

(j) When concrete attained a cube strength of not less than 3,000 p.s.i., bars 9, 10, 11, 12 and 13 were stressed across the joint.

(k) Deck slab concreted from B to D.

(l) When the deck slab from C to D attained a cube strength of 3,000 p.s.i., bars 18, and 19 in the beam and 15, 16, 17, 24 and 25 in the deck were stressed.

(m) Temporary pier was then removed and placed at the next span.

(n) The remaining portions of the deck slab were cast and when the concrete attained the required cube strength, bars 4, 5, and 20 (20 in the outer beam only) in the beam and bars 14, and 23 in the deck were stressed. Bar 5 was restressed to make up losses due to elastic shortening.

(o) Finishes, footpaths, hand rails, etc., were then completed for this span.

The same procedure was repeated for all other spans except the suspended span where a slightly different arrangement had to be made.

In the case of the suspended span, after the beams were erected in position, diaphragms and complete deck slab were cast and when the concrete attained the requisite strength, the remaining bars were stressed and then finishes, footpaths, etc., were completed.

12.8. Stressing Procedure

The procedure described below was found to be extremely satisfactory and is recommended :

(a) *Stressing bars from both ends*

(i) The threads at both the ends of bars were thoroughly cleaned with a wire brush and oiled, where necessary.

(ii) Necessary number of washers were put on so that the length of the bar extending over the end plate on the washer at each end is exactly 4-3/8 inches.

(iii) The adapter was then run up as far as possible so that the minimum length of the thread in the adapter is $1\frac{3}{4}$ times the bar diameter (in this case 1-31/32 inches). The adapter should run very freely on the bar to indicate that there was no cross-threading.

(iv) The jack was moved up with the piston in the forward zero position and the cotter was put home. The release valve was then closed and the jack was pumped until it was correctly seated on the end plate and had just begun to take load. The zero extension in the vernier was then read. Pressure was increased at the rate of 5 tons at each stage for both ends simultaneously and corresponding extensions were noted to see that extension at both ends was almost equal. If not, more pressure was put at the end having lesser extension. After the total extension was read, the gauge readings were noted and verified with those of the theoretical ones. In almost all cases, the pressures were found to be within plus or minus 5 per cent of the theoretical values. At every stage, watch on the gauge readings was kept in order to see whether the stressing operations were in order. Any excessive elongation without corresponding rise in jack pressure would indicate that the bar was jammed somewhere. A sudden fall in the jack pressure would indicate that either the concrete had failed or the rod had snapped. If the gauge pressure increased with jerks, it would indicate excessive frictions at some points which were being overcome during stressing.

(v) When the desired total elongation at both ends was achieved, the nuts were followed up with ring spanners and tommy bars and were finally tightened until the needle of the pressure gauge flickered, indicating that the nuts were taking up the load. The jack was then released gradually and removed to the next position, Photo 8.

(b) *Stressing bars from one end*

(i) Threads at both ends were thoroughly cleaned with a wire brush and oiled, where necessary.

(ii) One standard washer 3/16 in. thick and a nut were put at the end where the jack was not being fixed. The nut was then tightened firmly till it was driven completely home with a tommy-bar. The other end was held fixed by means of the jack so as to prevent rotation of the bar.

(iii) This end was next pushed till the washer was firmly seated against the end plate.

(iv) At the jacking end, necessary washers were added so that the projecting thread length was exactly 4-3/8 in. When more than 1 in. thickness of washers was needed, it was decided to transfer some thread length on to the non-jacking end.

(v) The nut was then cleaned and put on and tightened by a ring spanner till it was driven completely home. The adapter was then run as far as it went, but in no case less than $1\frac{3}{4}$ diameter, *i.e.*, 1-31/32 in., was infringed upon.

(vi) The procedures (iii) and (iv) of (a) above were repeated.

12.9. Some Essential Precautions

(a) The only purpose of provivding the sheath was to prevent contact between the high tensile roads and the concrete. Hence, before putting up both the side shutters, extreme care was taken to check whether the sheathings were damaged anywhere. If so, repairs were done with black tape.

(b) Particular care was needed in placement and compaction of concrete round the end plates to make certain that there was no spongy concrete.

(c) Extreme care should be taken in working the needle vibrators as otherwise the sheaths might be damaged causing ingress of mortar inside the ducts.

(d) The jack trolley must be placed clear of the line of stressing and no person should be permitted to stand behind the jack.

(e) End plates should always be fixed normal to the direction of stress. This is checked by running a nut upto the end plate and checking the gap.

(f) The weight of the jack at all times must be taken by the winch and no weight should fall on the bars which might cause bending of the same.

(g) Proper record of prestressing of the bars of the different beams, should be carefully maintained. A typical record of prestressing of bar 19 of beam No. I_1 A_1 is given in Table 9.

12.10. Grouting Procedure

The aim was not to leave any bar ungrouted for a long period in a stressed condition.

For the purpose of grouting, the ducts were flushed with water under pressure to wet the sides. The excess water was removed from the ducts by compressed air. Cement was sieved through a 200 mesh sieve to remove any lumps or large particles which might cause a blockage during injection. Cement and water were measured by weight keeping water-cement ratio between 0.40 to 0.50. Water was added gradually to form a thick slurry. The slurry was then poured in the grouting machine and it was forced through the duct by compressed air working at a pressure of 80 p.s.i. until grout was expelled from the other end. That end was then sealed and the pressure was built up in the cavity before sealing the grouted end. During the process of pumping, the grout was kept well stirred to prevent initial setting. The ends were plugged with wooden plugs which were left for a day or so. When all the ducts were injected in this way, wooden plugs were taken out, mild steel bars provided for protecting concrete behind the end plates reset and finished with 1 : 2 cement mortar.

TABLE 9

Prestressing chart

Beam No I_1 A_1 Bar No. 19. Date of prestressing 8-8-60

Bar end A articulation				Bar end C (constn. joint end)						
Gauge in ton	Vernier reading	Extn. over load range in inches	Total extn. in inches	Gauge reading in tons	Vernier reading	Extn. over load range in inches	Total extn. in inches	Total extn. of bar in inches	Designed extn. in inches	Remarks
0	0·300			0	0·244					
5	0.466	.166	0.166	5	0.400	.156	0.156			
10	0.600	.134	0,300	10	0.533	.133	0.289			
15	0.866	.266	0.566	15	0.688	.155	0.444	2.91	2.90	
20	1.000	.134	0.700	20	0.844	.156	0.600			
25	1.233	.233	0.933	25	1.000	.156	0.756			
30	1.400	.167	1.100	30	1.155	.155	0.911			
35	1.600	.200	1.300	35	1.322	.167	1.078			
40	1.688	.088	1.388	40	1.511	.189	1.267			
42	1.733	.045	1.433	44	1.677	.166	1.433			

Add initial extension due to compression of concrete } 0.022 / 1.455   0.022 / 1.455

In the above case, the required elongation was 2.90 in. and the observed elongation was 2.91 in. which was satisfactory. Friction losses were calculated according to Morrice & Cooley.

13. ERECTION OF BEAMS

13.1. As already stated, the adjacent span and support beams were to be placed temporarily over a temporary support for jointing and casting the diaphragms, deck slab, etc. As the temporary pier was to support about 125 tons of load, it was properly designed. It was to function till the unit became self supporting.

13.2. The temporary pier designed for supporting 125 tons load composed of 6 numbers of steel stanchions, each comprising one number of 12 in. × 5 in. R.S.J. resting on 2 numbers of 12 in. × 5 in. R.S.J. which, in turn, rest on road bearers 9 in. × 9 in. × 1/4 in. with sand filling, in between. The longitudinal central line of the temporary pier was designed to coincide with the longitudinal central line of the bridge whereas the transverse central line of the temporary pier was in line with the face of the span beam. The foundation grillage consisted of 28 numbers of road bearers placed side by side in the longitudinal direction supporting the stanchion base, which was connected to the road bearers by means of cleat angles and base plate 21 in. × 21 in. × 5/8 in. thick. The stanchions were suitably braced with $2\frac{1}{2}$ in. × $2\frac{1}{2}$ in. × $\frac{1}{4}$ in. M.S. angles. The seats for the beams over the temporary pier were made with 2 numbers 15 in. × 5 in. R.S.J., 12 ft 6 in. long, bolted to the top of the stanchion keeping a gap of 12 in. Timber packings were placed under the soffits of the beams to fit the curvature. The standard height of the temporary pier was 20 feet from the base of the stanchion to the top of the seats. To suit the fixed level at which the precast beams were to be seated, deep excavations were sometimes necessary due to the extreme undulation of the river bed. Since deep excavations in such close proximity of the casting bed might lead to damage of the precast beams, the arrangement was changed to salballah trestles, Photo 1.

13 3. There was practically no load on the temporary pier from the support beams except the unbalanced working loads and deck slab load. Since the total load to be carried by the temporary pier was only 125 tons, which was spread over a sufficiently large area, shallow foundations were adopted in the dry portion of the river bed. Where the foundations were to be placed in flowing channel, salballah pillings were resorted to as a precautionary measure against small scours. However, such foundatians were only a few. In other places, where the velocity and depth of water were not much, 2 ft 6 in. boulder soling was laid under the base and the sides were protected by a ring wall of gunny bags filled with boulders for protection against possible erosion.

13.4. Precast beams after being prestressed were lifted up from their casting beds by means of slings attached to the beams approximately 7 ft from either end in case of span beams and 5 ft from the centre line in case of support beams. The slings were fixed to pulley blocks hung from the gantry. The beams were hoisted by crab and winch and then laterally pulled by means of travelling trolley mounted on gantry girders. Photo 9 shows hoisting of the beam in position.

13.5. Trestles supporting the gantry girders were made of four steel columns having 12 ft by 8 ft base. Each trestle was composed of 4 in. × 12 in. × 5 in. R.S.J. stanchions braced with $2\frac{1}{2}$ in. × $2\frac{1}{2}$ in. × 1/4 in. M.S. angles. The 'N' type gantry girder was 46 ft long, 4 ft deep and made of angles and channels. Maximum designed load at the column base including its own weight was calculated as 32 tons. Since there was no other load except the vertical weight of the beams to be handled, the stanchion base was made of 4 ft 6 in. × 3 ft × 9 in. deep R.C.C. precast blocks resting on well rammed sandy base. The wind force was taken care of by guy ropes attached to dolphins driven 10 ft into the ground. The height of the trestle was so fixed that there was adequate clearance between the top of the beam and the bottom of the gantry girders.

The clearance below the overhead girders was kept 14 ft over the seats of the temporary pier, thus giving a margin of about 6 ft over the moving units. Some adjustments were necessary on account of undulations of the ground level. This was done either by excavating the ground or by placing precast R.C. sleeper cribs.

13.6. Lifting arrangement of the beams from the casting yard was originally planned with 15 ton chain-pulleys mounted on trolleys. But, during actual design, dimensions of the girders had to be changed causing an additional weight of 3 to 4 tons, which the chain pulley blocks were unable to carry when the first shore span beam was being erected. The teeth of the worm gear were badly damaged and the pulley block had to be rejected. Later on, the lifting was done with the help of crab and winch. Mechanical advantage was taken by using a differential pulley block which reduced the mechanical effort from 18 tons to 3 tons.

13.7. While lifting of the beams was done by means of crab and winch, its lateral shifting and lowering in position were done by the trolleys moving over the bottom flanges of 15 in. × 6 in. R.S.J. gantry beams bolted to the bottom flanges of the N girders. The trolley was a small one with a central pinion, the shaft of which could be revolved by operating a chain wound round a drum attached to its end. The pinion in turn drove two worm gears which had rollers attached to the shafts of the gears. The rollers rolled over the bottom flange of the gantry beam and thus imparted the lateral movement to the precast beams. Due to mechanical advantage, the trolleys loaded with the beams could bé moved easily by six labourers at each end.

13.8. For picking up the beams from the casting bed, special cradles were fabricated. These cradles were made of M.S flat 9 in. × 3/4 in. bolted at the bottom to a double channel 8 in. × 3 in. × 1/2 in. supporting the bottom flange of the beam. The top of the cradles consisted of a M.S plate 9 in. × 3/4 in. with a 4 in. diameter hole for fixing the lifting pulley. The open spaces between the cradle and the beam were packed with timber pieces and wedges to prevent wobbling during lifting.

13.9. After lifting the beam to the required elevation, it was moved laterally by the travelling trolleys and when it reached correct position, the beam was lowered down so as to sit on the bearing. It was also made certain that the gap at the construction joint was exactly 2 ft. This space was required for coupling the rods. After the beams were lowered into position, diagonal struts were fixed for preventing overturning.

(j) The hoisting arrangements were made very simple so that ordinary workmen could handle the same with ease. However, the first accident occurred when the imported chain pulley block failed. The second one took place when one of the towers supporting the gantry trusses fell down on account of a heavy gale passing over the work site with a speed of 80 m.p.h. This damaged tower hit the end of one of the support beams in position and then fell down on one of the span beams lying on the beds ready for hoisting. The beam got tilted and struck an adjacent beam which fell on the ground and broke at one place making the beam unserviceable. However, it was fortunate that no one was injured. The work of salvaging the other beam which was not damaged, was continuing at the time of writing this Paper.

14. CONCRETING OF CROSS BEAMS AND DECK SLAB

In this connection, there is nothing very much interesting to report excepting the arrangement for raising the concrete from the river bed to the deck level. For this purpose, a mechanical hoist 25 ft high was erected.

The hoist consisted of a vertical frame made of M. S. angles and plates. The inside face which carried the bucket was greased. Steel rollers were fixed to the sides of the bucket which was lifted through the hoist by means of a power winch. When lifted to the required elevation, the bucket was moved laterally and concrete released by opening a latch. This improvised mechanical arrangement was found to be quite satisfactory. On an average, about 150 cu. ft. concrete could be carried to a height of 25 ft per hour.

15. CONCLUSION

It will be evident that the system of deck structure described in this Paper has two inherent disadvantages, namely, (i) the construction must proceed from both ends of the bridge, and (2) if any one span gives way, then all the spans in between that span and the suspended span, including the suspended span would fall down. As such, its use, though economical over simply supported spans, should not be adopted in road system of strategic importance.

ACKNOWLEDGMENTS

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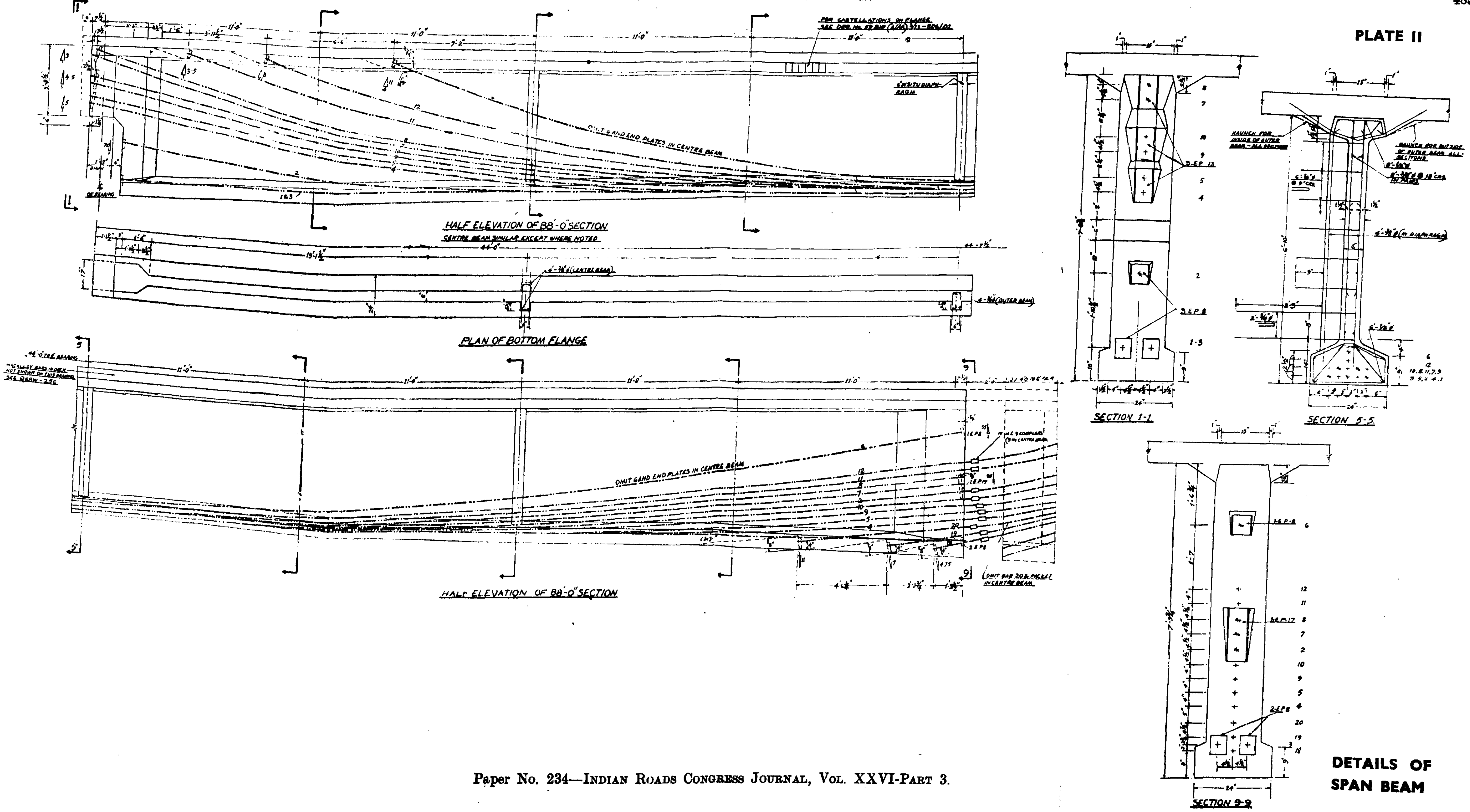
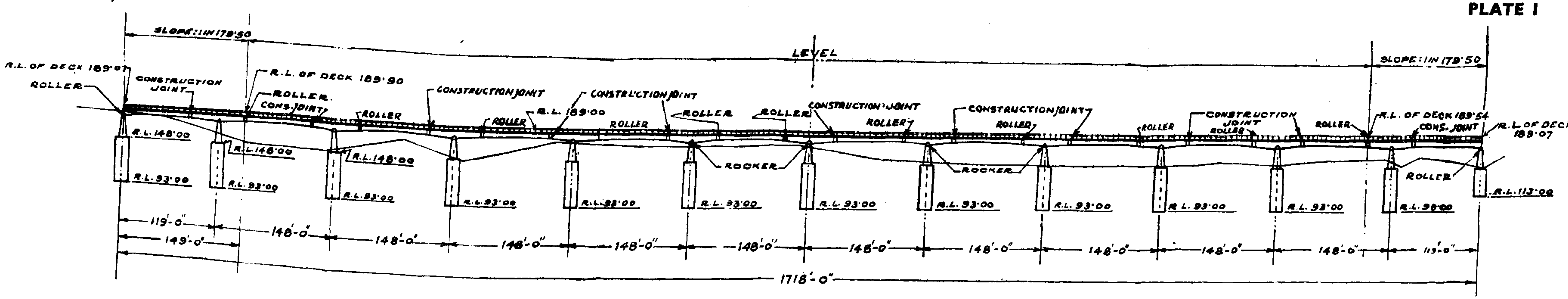
PLATE II

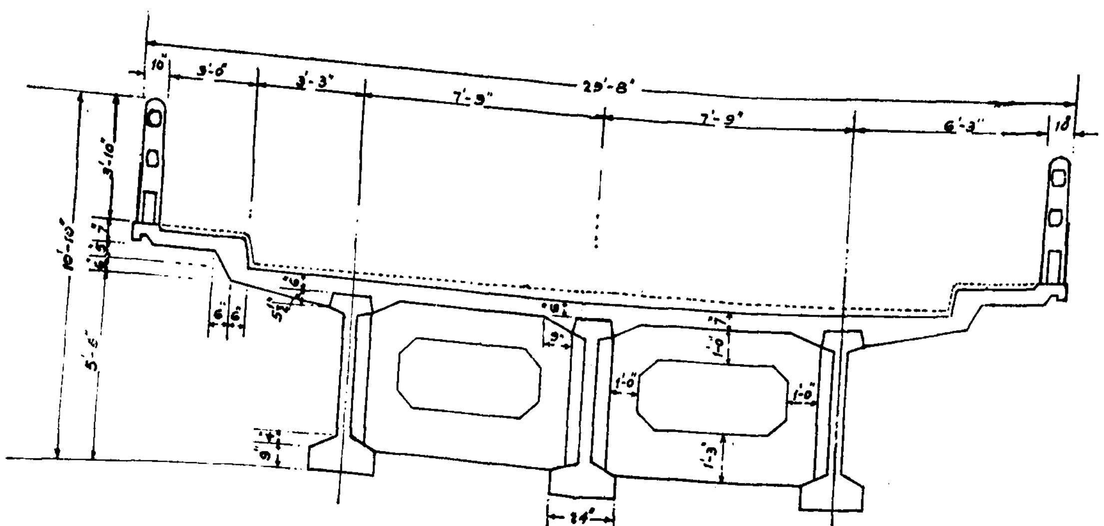


DETAILS OF SPAN BEAM

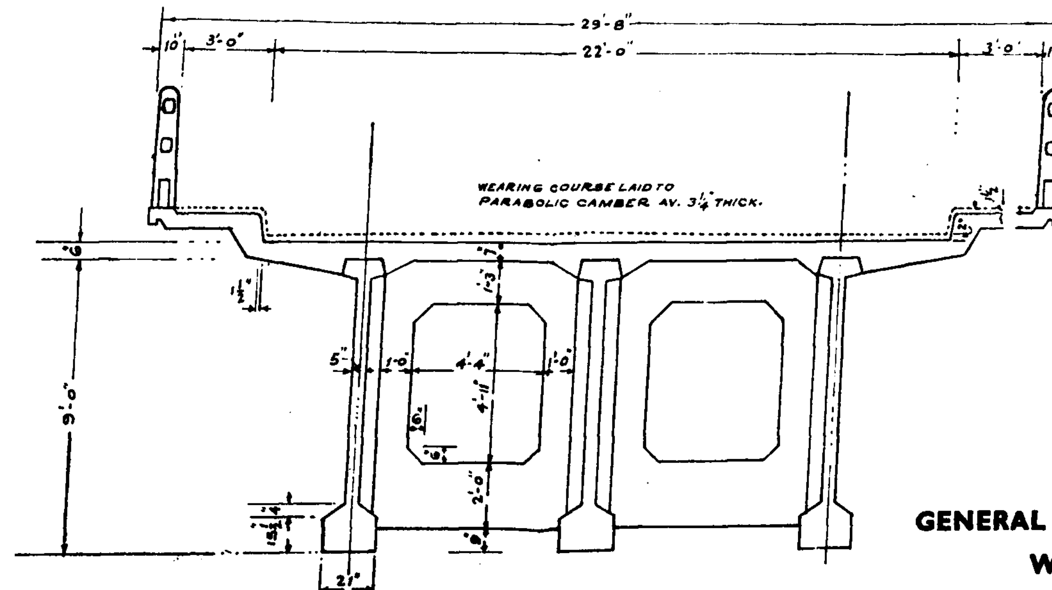
PLATE I



PROPOSED ELEVATION AS PER REVISED DESIGN
(ACCORDING TO DISCHARGE OF 3,25,000 CUSECS.)
NOT TO SCALE.



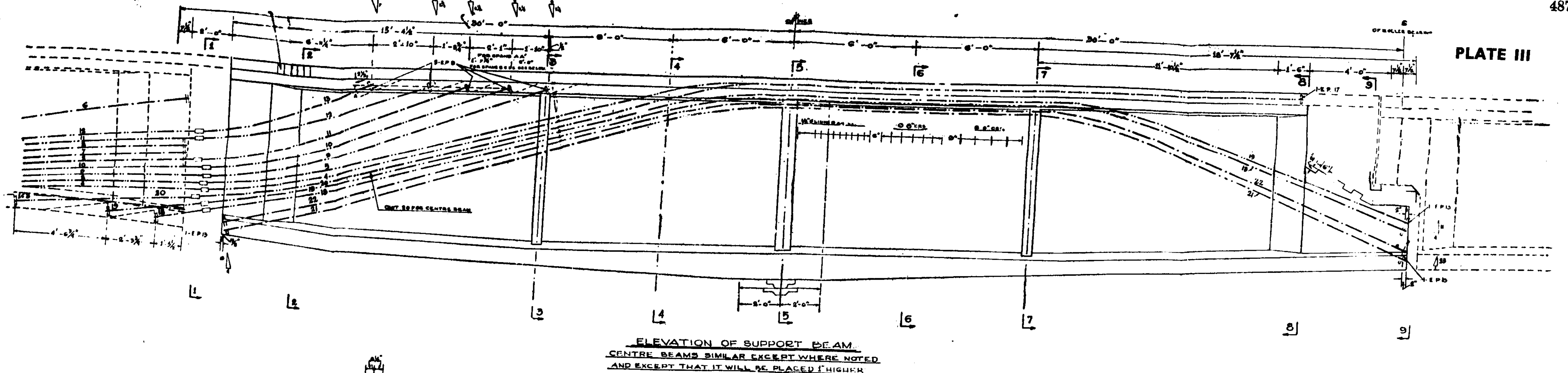
MIDSPAN SECTION.
NOT TO SCALE.

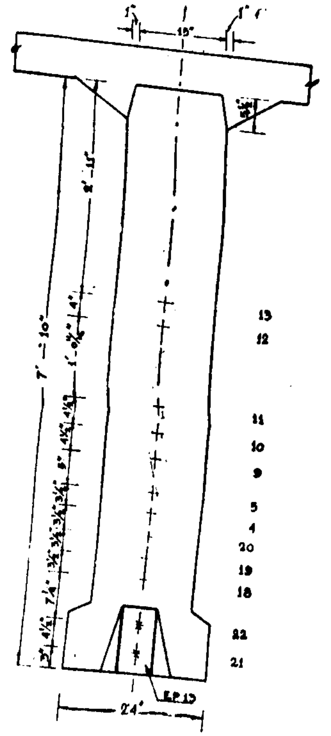


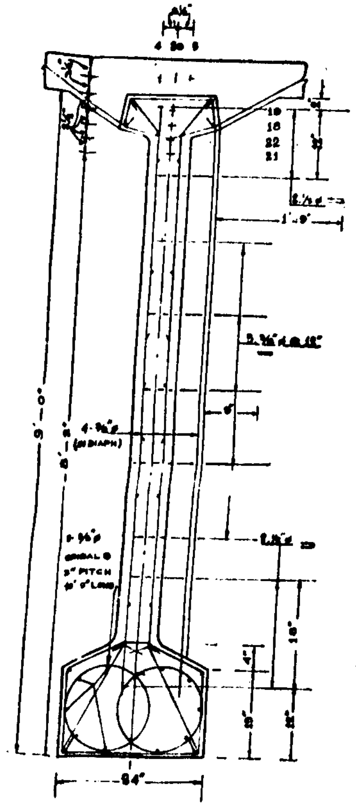
SUPPORT SECTION.
NOT TO SCALE.

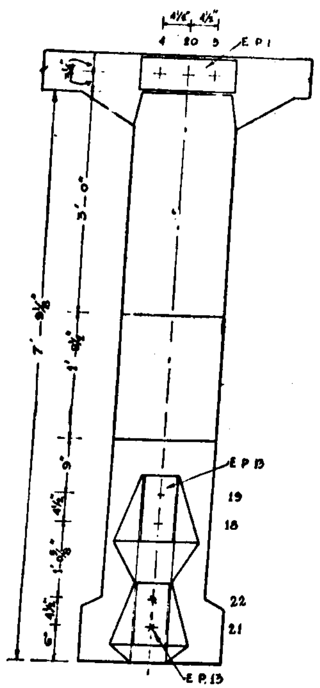
GENERAL ELEVATION WITH CROSS SECTIONS

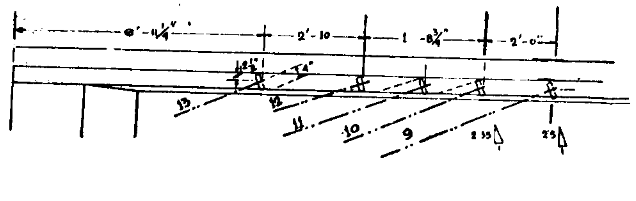
PLATE III







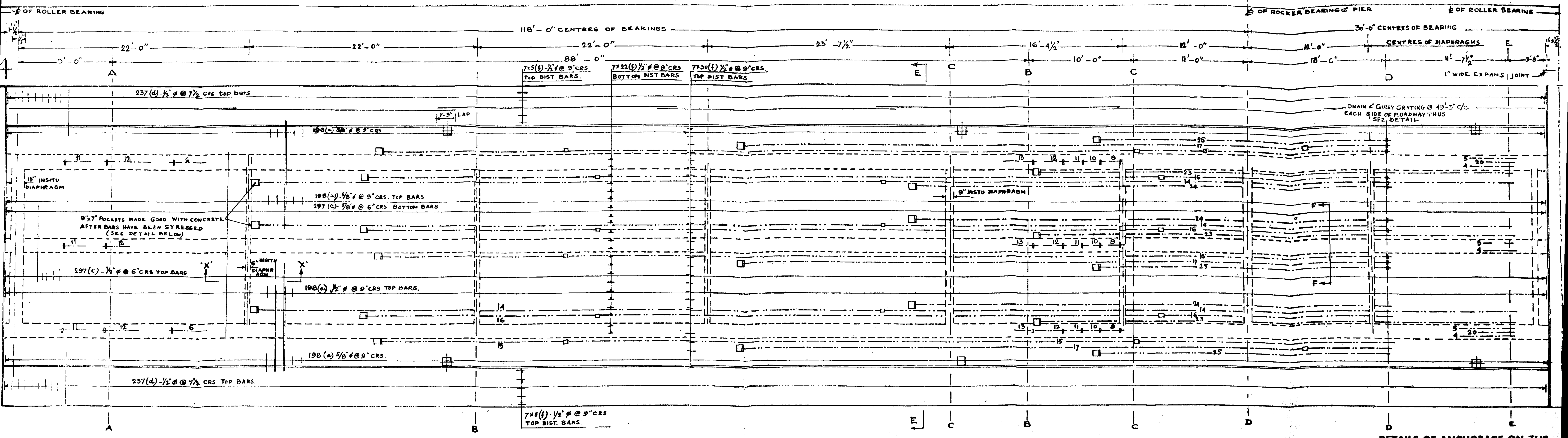




DETAILS OF SUPPORT BEAM (OUTER)

GENERAL REPORT
ON
ROAD RESEARCH IN INDIA
1960-61.

PLATE IV



DETAILS OF ANCHORAGE ON THE DECK SLAB

" GENERAL REPORT ON ROAD RESEARCH IN INDIA, 1960-61* "

CONTENTS

INTRODUCTION

A review of the Road Research carried out this year by the various Research Organisations in the country indicates that main emphasis has been laid on applied Highway Research. Many systematic investigations for improving the existing roads, as well as for construction of new roads are reported to have been carried out. The study of soft aggregates for use in road construction has also received considerable attention.

Some fundamental work too has also been reported by Central Road Research Institute to explain the chemical reactions of clays calcined at different temperatures.

This report has been compiled on the basis of Reports received from the following Research Organisations :

1. Central Road Research Institute, New Delhi.

* Compiled by the Director, Central Road Research Institute, New Delhi-20.

2. Engineer Research Wing of the College of Military Engineering, Kirkee.
3. Highways Research Station, Madras.
4. P.W.D. Research Institute, U.P., Lucknow.
5. B & R Research Institute, West Bengal.
6. B & R Research Laboratory, Punjab.
7. Engineering Research Department, Andhra Pradesh.
8. Mysore Engineering Research Station, Krishnarajasagar.
9. P.W.D., Gujarat State.
10. P.W.D. Maharashtra State.

It is likely that there may be some omissions due to some of the reports not having been received in time for compilation of this report. It is, however, hoped that such points will be brought up during discussions so as to make the report as comprehensive as possible.

After briefly reviewing the work done, salient points are given at the end of each section to facilitate discussion.

I. DESIGN, CONSTRUCTION AND MAINTENANCE

1. RIGID PAVEMENTS

1.1. Design

General

In continuation of the work on measurement of Temperature differentials in concrete pavements, being carried out by the Central Road Research Institute[1]; cement concrete slabs 14 ft × 12 ft of 4 in., 6 in., 8 in., 10 in., 12 in. and 16 in., thicknesses have been cast in the premises of Central Road Research Institute. Temperature measurements at different depths were recorded by automatic continuous recorder, using thermo-elements. Similar master-stations are proposed to be set up in different zones of the country for collecting data during the peak temperature differential and peak temperature period. Arrangements are being finalised with the Chief Engineers of Punjab, West Bengal, Maharashtra, Andhra Pradesh, Madras and Kerala for completing in their States at Jullundur, Calcutta, Poona, Hyderabad, Madras and Trivandrum respectively, construction of similar slabs 10 ft × 10 ft and of 4 in., 6 in., 8 in., 10 in. and 12 in. thickness for this investigation.

1.2. Construction and Maintenance

1.2.1. Field Experiments

Pavement for tracked vehicles

In continuation of previous investigations, the College of Military Engineering has reported further work on the effect of tracked vehicles on cement-concrete and stone-set pavements, conducted by subjecting them to 20 straight and 20 slewing passes of a tracked vehicle. The data obtained so far is under study; to obtain more information it is proposed to continue these tests and to subject the pavements to 200 passes—straight and slewing.

Bonded Concrete Overlay Test Track

Based on the laboratory work done on overlays at Central Road Research Institute, as reported last year, an experimental stretch, one furlong in length, on the existing 3 in. thick cement concrete road between Hapur—Garhmukteshwar has been provided with a bonded concrete overlay.

The required thickness of monolithic concrete pavement designed for k-value obtained by plate-bearing tests on base course; using Westergaard Formula as modified by Teller & Sutherland for load stresses, and Thomlinsons analysis for temperature stresses, was found to be 7 in.

Assuming that it will be possible to create perfect bond between the overlay and the existing slab, it was decided to provide a 4 in. thick overlay slab so as to make up the 3 in. thickness of the existing pavement to the required thickness of 7 in.

For obtaining bond between the existing pavement and the overlay thereon, the following procedure was adopted. The existing surface was thoroughly cleaned of all dust, traces of bitumen, etc., and was then treated with commercial hydrochloric acid diluted with equal volume of water, at the rate of 7 lb per hundred sq. ft. After the effervascence stopped the surface was washed copiously with water to remove all traces of acid. On the dried surface, a thin slurry coat of 1 : 1 cement-sand mortar was given just in advance of concreting operations for the overlay.

As the joints in the existing pavement were 33 ft apart in the overlay, expansion joints were provided 99 ft apart and intermediate dummy contraction joints at 16½ ft apart.

On a section with cracked old pavement, to assess the efficacy of reinforcenent in prevention of reflection cracking in the overlay, M. S. reinforcement was provided at the rate of 8 lb per square yard in the overlay.

It is proposed to cut out cores from the test-track for conducting bond strength between the existing pavement and the overlay.

Quality Control in Pavement Construction

Concrete mix used for the overlay was designed for 650 p.s.i. flexural strength. By quality control by the field laboratory, and use of small size swing type weigh batcher, it was possible to achieve a field mix with a coefficient of variation in flexural strength of the order of 10 per cent only.

1.3. Field Surveys and Pilot Plant Studies

1.3.1. All India Survey of Puzzolanic Clays

The Central Road Research Institute has completed its survey of major clay deposits in Rajasthan State for the industrial production of reactive puzzolanas for use in mortars and concrete. Out of the 33 deposits investigated, it was found that twelve could yield reactive puzzolanas. However, for exploitation purposes, many factories, such as the nearby location of cement factories, lime kilns, or other important engineering construction works, the access to the deposits, facilities of tranpsort, etc., have to be taken into consideration.

1.3.2. Pilot Plant Experiments on Industrial Production of Reactive Surkhi

The Central Road Research Institute has carried out limited experiments on the controlled calcination of clays through the courtesy of Shri Ram Institute, New Delhi, using their oil-fired rotary kiln with a view to produce a reactive surkhi that will give a specified minimum lime reactivity strength. The internal diameter of rotary drum was 15 in. and its length 10 ft. Depending upon the rate of charge, the speed of rotation and the time of travel of the material and the temperature gradient within the kiln, it was found that the temperature of firing at the discharge end could greatly exceed the optimum temperature determined in the laboratory on small, stationary samples using a muffle furnace. The exact conditions of operation for each kiln and material for optimum production have to be evolved through experimentation. For the particular kiln and the type of material used, it was found that a production of about 5 tons per day at a cost of Rs 40 per ton of ground surkhi was possible. Further experiments with improvements in the kiln and with other types of kilns are proposed to be undertaken.

1.4. Laboratory Research

1.4.1. Admixtures in Concrete

Use of Clinker and Fly ash in Concrete:

The Highway Research Station, Madras, has reported the results of further tests with a local fly ash both as a replacement and as addition to cement in mortars and concrete[3]. The results show that while in mortars there is little fall in 28 days' compressive strength for mortar cubes upto a cement replacement of 10 per cent, the concrete cylinders showed some reduction in strength at all percentages of replacements.

The Railway Testing and Research Centre at Lucknow, has reported the results of their extensive investigations on the use of the loco coal ash as a puzzolanic replacement of cement in concrete[5]. It is shown that with a 20 per cent replacement of the cement, the same strength of the plain concrete without imparing any of the other properties of the concrete could be obtained by reducing the water content and adjusting sand content. A procedure for making the mix design adjustments has also been evolved.

Surkhi as a Puzzolana

The reactivity of surkhis prepared from loamy soils in U.P. was investigated by the P.W.D. Research Institute, Lucknow, starting with 3 soils with a clay content of about 25, 29 and 30 per cent[6]. Two of these soils are clay-loams near the border-line of clay and the third is a clay as per the triangular classification. Of these the clay-soil and one of the clay-loams passed the I.S. Specification of lime reactivity test with values of 845 and 765 p.s.i. respectively while the other clay-loam failed, with a lime reactivity of 403 p.s.i.

The Road and Building Research Intitute, West Bengal has reported an increase of about 6.7 per cent in the 28 days compressive strength for concrete mixes with 30 per cent surkhi replacement of cement[7].

The Central Road Research Institute has carried out some fundamental work on pure and mixed oxide systems that can throw some light on puzzalanic reactions[1]. It has been revealed for the first time that certain parallel relations exist between the requisite structural conditions for the effective functioning of a Silica-Alumina cracking catalyst used in petroleum industry and reactive puzzolana of the alumina silicate type. These include the tetrahedral co-ordination of the Aluminium, the strain in the bonds and the 'acid' character imparted to the silica alumina by the AlO_4 part,

Air-entrained concrete for Road Construction

The P.W.D. Research Institute, Lucknow has undertaken a study with a view to evolve suitable mix design for air-entrained concrete, using the Rihand Air Entraining Agent as manufactured by Rosin Co. Ltd. Bareilley. Work is reported to be in progress.

1.4.2. Test on Cement and Concrete

Relationship between Water/Cement ratio and Compressive strength of concrete made with Indian Cements

In continuation of the previous work, two more brands of cement were investigated at the Central Road Research Institute[1], bringing the total number of cements so far investigated for this purpose to six. Studies completed so far indicate that even though all the cements studied comply with the standards laid down by the I.S.I., if the same were used in concrete mixes, keeping all other factors constant for a particular water cement ratio, compressive strength obtained in some cases may vary by as much as 75 per cent.

Partial Replacement of sand by soil in Cement Mortar

The Madras Highway Research Station has continued its last year's studies with a second soil which is a sandy loam with 56 per cent sand, 36 per cent silt and 6 per cent clay[3], and has a liquid limit of 28 and plasticity index of 15. Different percentage of sand was replaced by this soil in a 1 : 4 cement sand mortar using standard sand. The results indicate that

(1) the compressive and tensile strength of the mortars improved with the partial replacement of sand by this soil upto 30 per cent, and 20 per cent respectively,

(2) there is a general increase in workability of the mortar with increasing replacement of sand, for the soil investigated.

Tests on the effect of sand replacement by soil on the durability of the mortar cubes are in progress.

Correlation of Compressive, flexural and split tensile strengths of concrete

The Madras Highway Research Station has reported[3] the results of their investigation using a 1 : 2 : 4 concrete with water cement ratio of 0.55, on 6 in. dia × 12 in. high cylinders for compressive and split tensile strength tests and on 6 in. × 6 in. × 28 in. beams for flexural strength in both cases 28 days after casting. A statistical analysis of the results revealed the following relationships.

Flexural strength = 3.59 × Cylinder strength 0.6244 Split tensile strength = 1.91 × Cylinder strength 0.6723

Assuming the cylinder strength to be $\frac{3}{4}$ the cube strength, similar relationships, between flexural strength and cube strength as also between split tensile strength and cube strength have been derived. The later relation i.e. the relation between split tensile strength and cube strength is shown to tally well with results obtained from similar studies in and outside the country.

Strength of Mortars and Concrete with different aggregates

The P.W.D. Research Station, Lucknow, has studied the effect of using (1) a local fine sand (Gomti sand), (2) A coarse sand (Banda sand), (3) stone ballast (Jhansi grit) and (4) overburnt brick ballast previously soaked and without soaking, on the strength of concrete[6]. The mixes tried were 1 : 2 : 4 or 1 : 3 : 6.

The results also reveal that the concrete with soaked brick ballast gave much higher strength than made with the brick aggregate without soaking. Comparison of the 1 : 2 : 4 mix using Banda sand and Jhansi grit with that using Banda sand and soaked brick ballast reveal that even though the strength of the latter is only about 80 per cent of that of the former, it is feasible to get satisfactory concrete using overburt brick ballast in place of stone grit in areas where good stone aggregates are not available at reasonable cost.

The P.W.D. Research Station, Lucknow, has also investigated the suitability of some locally available fine sands as such or after mixing with coarser sands in masonry mortars for some bridges[6]. The coarser sands always gave better strengths than the local sand and the admixture of the two usually resulted in some improvement of the local sand, the extent of improvement varied somewhat with the proportions employed and the individual characteristic of each sand.

Mix design for lean concrete base course

Experiments connected with the laying of lean concrete bases with black top at Mount Road in Madras, are described by the Madras Highway Research Station. A 1 : 16 mix using a coarse aggregate grading of $66\frac{2}{3}$ per cent of 1 in. and $33\frac{1}{3}$ per cent of $\frac{1}{2}$ in. material has been recommended.

1.4.3. Alkali Aggregate Reaction

In continuation of the previous work which indicated that certain microcrystalline or highly fractured or internally strained quartzites could exhibit alkali aggregate reaction in concrete, fifty aggregate samples from different quarries comprising mostly of quartzite, trap, siliceous lime stone and sand stone were chemically examined, for dissolved silica and reduction in alkalinity, by the Central Road Research Institute.[1] Some of these including a few quartzites showed possibilities of Alkali Aggregate Reaction. Further confirmatory work using mortar bars is in progress.

1.4.4. Resistance to Sulphate Attack

Previous work at the Central Road Research Institute showed that clay calcined at a temperature of 1000°C imparted better resistance to concrete when used as a partial replacement of cement compared to clays calcined at lower temperatures. A plausible explanation for this phenomenon has been suggested.[8] At 1000°C, the released Al_2O_3 from any clay after its structural disintegration is in the form of the high temperature gamma alumina or alpha alumina, both with a comparatively better crystallinity, ordered structure and reduced reactivity. At lower temperatures, the alumina is in the highly reactive, microcrystalline, low temperature form of gamma alumina with a defective structure. In the system $CaO—Al_2O_3—CaSO_4—H_2O$, marked reaction leading to Ettringite formation could take place only with the reactive form of gamma alumina as obtained from clays calcined at lower temperatures. Hence Ettringite formation and the consequent expansion and cracking of concrete could occur with clays calcined at lower temperature while the above reactions do not occur with those burnt at higher temperature. Experimental work is being undertaken to verify the validity of the above hypothesis.

1.4.5. Use of Blast Furnace Slag in Road Construction

Work at the Central Road Research Institute on representative samples of blast furnace slag from the Hindustan Steel Ltd. have indicated that the slag could be used in base course, wearing course as also as an aggregate in concrete.[1]

1.4.6. Joint Sealing Compounds

Continuing the previous experiments the College of Military Engineering tried a bitumen based compound with a top coat of tar for joint sealing.[2] This was found to be unsatisfactory. Compounds based on pitch and road tar with 25 per cent bitumen and less, mixed with synthetic rubber with plasticisers are being experimented.

1.4.7. Expansion Joint Filler from Coconut Pith

The Central Road Research Institute has continued its studies on the economical production of premoulded expansion joint fillers by binding coconut pith into sheets.[1] Trials were made on hot pressing the mixture of coconut pith, rubber latex with certain vulcanizing materials and accelerators. Experiments with reduced latex content have given a product satisfying the requirements of recovery, compressive strength, extrusion and weather durability according to the A.S.T.M. designation D 544-49. The finished

product produced appears to be competitive both in cost and in quality. Final investigations working out the practical details of production which could be undertaken by the coir industry are in progress.

1.4.8. Brick Concrete Pavement

With a view to develop an economical type of road structure, Central Road Research Institute has commenced a study on evolving a brick concrete pavement in which the central zone in the cross-section of concrete in a concrete road is proposed to be replaced by bricks in view of lower order of stresses developed in the central zone as compared to the extreme fillers.

Laboratory experiments on the compressive and flexural strengths of composite concrete specimens with brick cores are in progress. The results so far obtained on 8 in. cubes and 6 in. × 6 in. × 30 in. beams with replacement of central 50 per cent of concrete by bricks, show a loss of only about 5 per cent both in flexural and compressive strengths of concrete.

Based on the results of laboratory experiments, suitable specifications are proposed to be evolved for construction of an experimental field test track of brick-concrete pavement.

Sailent Features for Discussion

(1) Use of lean concrete bases.
(2) Use of brick aggregates in concrete.
(3) Industrial production of reactive surkhi.
(4)
(5)
(6)

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2. FLEXIBLE PAVEMENTS

2.1. Bituminous Pavements

2.1.1. Design

(i) Pavement Performance

From the reports received this year it is encouraging to note an increasing trend towards scientific field investigations into causes of failure of existing pavements for determining remedial measures for reconstruction. Many investigations both on flexible and rigid pavements, have been carried out by different research laboratories in the country, during the year under review.

2.1.2. Construction and Maintenance

(i) Field Experiments

(a) Observations on the performance of the various specifications with different quantities of bitumen and aggregate laid on an experimental test-track, conducted by C.R.R.I.,[5] during May-June, 1959, are in progress.

(b) C.R.R.I.[5] has constructed a test-track on the Delhi-Mathura Road to study the following:

1. Use of anti-stripping agents in surface dressing and in premix works using binders; bitumen 80/100 and Road Tar, RT-3; the anti-stripping agents used are: Doumeen-T, Fetro (developed in C.R.R.I.) and Turkey red oil and lime.
2. Use of precoated chippings for surface dressing; precoating was done both with 1 per cent RT-3 and bitumen 80/100.
3. Use of rubber in bituminous surface dressing with 80/100 bitumen.
4. Utilisation of low-temperature tar as a road binder. The binders processed from low-temperature tar were used for surface dressing and premix works.

   The test-track was laid in May, 1961 and is under observation.

(c) The experimental stretch on the Mount-Poonamallee Road, laid by the Highways Research Station, Madras[4], using the additive, "Adhevia-T" with both wet and dry chips continued to be under observation for the behaviour of the surface under traffic.

Mastic Asphalt Pavement

(d) The Military Engineers experimental wing of the College of Military Engineering, Kirkee[6], have laid a mastic asphalt pavement partly on an existing bituminous road and partly over an existing concrete pavement. Different grades of bitumen, viz., 10–20, 20–30, 30–40, 80–100 and 180–200 penetration were used in varying proportions with aggregates ½ in. down, fine sand washed and dried, passing 1/8 in. and mineral filler—limestone powder and cement—passing 200 mesh sieve. The entire mastic asphalt pavement was subjected to accelerated tracked load traffic. The results are stated to be under study.

2.1.3. Laboratory Research

(i) Dense Bituminous Mixes

Different aspects of the design of bituminous concrete are receiving attention of various highway organisations.

In the C.R.R.I., investigations were continued on the design of dense graded asphaltic concrete mixtures[5]. Research work on the possibility of using silty soil[5] as filler in dense asphaltic mixture was carried out. Silty soils with P.I. values ranging from 6–14, which cover a wide range of soils, were tried. It has been observed that silty soils having a P.I. upto 14 can be used as a filler for dense asphaltic mixtures.

Highway Research Station, Madras State[4], reported to have taken up a study to find the comparative merits of fillers such as cement, granite quarry dust, lime, flyash and their combinations in asphaltic concrete mixes. The study is said to be continuing.

Highway Research Station, Madras State[4], designed during the year under review asphaltic concrete mixtures for two different roads. Hubbard field and immersion compression tests were used in the design. The same tests were used also in the design of mix for the binder course of the heavy duty asphaltic concrete pavement for certain roads in Madras city.

Study on the swell and absorption characteristics of bituminous concrete mixtures was continued in the C.R.R.I.[5]. A rapid test for the determination of swell and absorption has been developed. The properties of the compacted mixtures are evaluated on the basis of an index of absorption/swell. Based on a series of tests, it was found that a temperature of 140°F with vacuum of 10 in. of mercury and duration of 30 minutes represents the best combination for determining the absorption, swell and reduction in stability of the mixture.

(ii) **Stripping of Bituminous binder**

With a view to improve the poor adhesive characteristics of some of the aggregates which could be used for road works, further investigation was taken up by C.R.R.I.[5] by using different types of anti-stripping agents, such as Doumeen-T, Adhevia, T, Wet Fix, Udol-522 as well as Fetro, developed in C.R.R.I.

P.W.D. Research Institute, Lucknow,[8] has reported that efficacy of Udol-522 was tried as an anti-stripping agent. It was found that addition of 0·5 per cent resulted in marked improvement, in the adhesion between aggregate and binder. Work with moist aggregate is in progress.

Highway Research Station, Madras State[4], continued their study on "Additive; for improving adhesive characteristics of aggregates". Additives taken for study include "Adhevia-T, lime and Turkey red oil, creosote, tar, Doumeen-T, creosote solution of Doumeen-T, waste oil and cashewnut oil. The degree of adhesion was determined by static water immersion test, chemical immersion test (i.e., modified Riedel and Weber Test) and immersion-compression test. The investigations are stated to be still in progress.

Highway Research Station, Madras State[4], have taken up a comparative study of various adhesion tests to evaluate a reliable method of test. The various tests considered for the study are (i) Riedel and Weber Test, (ii) U. 37 German Test, (iii) Modified Riedel and Weber Test, (iv) Plate Test, (v) Static Water Immersion Test, (vi) Immersion compression Test, and (vii) Immersion Wheel Tracking Test. The work is in progress for conducting these tests on different types of aggregates.

(iii) **Precoating of overburnt brick-grit**

Effect of precoating with tar and bitumen overburnt brick grit was investigated at P.W.D. Research Institute, Lucknow[8]. Various percentages of binder were tried. 3 per cent binder was considered suitable to coat the grit. In order to cut down the cost, the binder was thinned with an equal quantity of cheap mobile-oil. Further work indicated that precoating with 1 per cent binder thinned and 1 per cent mobile oil improved the quality of the grit considerably.

(iv) **Cutbacks**

In continuation of the study directed towards deciding suitable cutbacks for surface dressing works in different temperature zones, the C.R.R.I. conducted investigations for measurement of fall of binder temperature with lapse of time, at various pavement temperature, ranging from 15.5°C to 60°C[5]. It

was found that the fall of binder temperature in relation to time at any pavement temperature is exponential in behaviour, i.e., the fall of temperature is rapid during the first 15 minutes, and becomes very slow thereafter. Further work is in progress.

(v) **Prevention of foaming in bitumen and tar**

Road and Building Research Institute, West Bengal[3] carried out some investigations to find out the effect of several reagents such as lime, Adhevia-T, silicones and Turkey red oil for assessing their comparative merits as anti-frothing agents of tar and bitumen. It is reported that lime has insignificant effect on defrothing of tar and bitumen (containing 3 per cent water), whereas silicones, Adhevia-T and Turkey red oil can be utilised as antifrothing agent to tar and bitumen.

(vi) **Weathering and Oxidation of Road Tars**

This study by and large, concerning the evaluation of the degree of weathering and its effects on the physical characteristics of the Indian Road Tars, is being carried out by the C.R.R.I[5]. Laboratory accelerated weathering tests like thin film oven evaporation, pressure oxidation, Infra red and ultra violet irradiation test have been completed on three grades of road tars, namely, RT-2, RT-3, and RT-4. It was found that hardening of the binder was maximum by infra red radiation. Oven evaporation test was found to be very useful in determining oxidation characteristics of road tars. Further work is in progress.

Salient points for discussion

1. Use of anti-stripping agents to improve the adhesion between binder and aggregates.
2. Use of cheap filler material in dense bituminous mixtures.

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2.2. Water-bound Macadam Roads

2.2.1. Use of Cement Stabilized Moorum as filler in W.B.M.

The Road Research Wing of Mysore Engineering Research Station[1], Krishnarajasagar, reported construction of two stretches about a furlong each, in water bound macadam, where the moorum used for filler was stabilized with 5 per cent and 7½ per cent of cement by volume respectively, to improve binding quality of moorum and to observe the wearing quality of the resulting surface. It was stated that the moorum-cement mixture was cured for a week to allow development of strength; however, the traffic was allowed on the road during this period also. The stretches are under observation.

2.2.2. Study of Aggregates for use in Road Construction

Study of aggregates has received a good deal of attention from different research laboratories during the year under review. In addition to the work carried out at Central Road Research Institute [2],[3], on the testing of aggregates to assess their suitability for road work, similar studies have been reported by the States of Gujerat [4], West Bengal,[5] Uttar Pradesh[6] and Madras[7]. The tests generally conducted are for determination of specific gravity, water absorption, aggregate crushing values, Los Angeles abrasion value, Deval attrition value, and stripping.

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2.3. Soil Roads including Stabilization

2.3.1. General

During the year under review, there has been great enthusiasm among the engineers to use local soil and consequently longer lengths of roads have been constructed using stabilized soil. Pilot projects on the stabilization of soil with soft aggregates, which were sponsored by C.R.R.I.[1] were extended during the year to the States of Jammu and Kashmir, Mysore, Orissa, Kerala and Andhra Pradesh. Reassured by the success of these projects, the States of Uttar Pradesh and Rajasthan are taking up longer lengths.

The use of processed soil as a part of the designed thickness of road pavement has been taken up by the Uttar Pradesh State P.W.D. for construction of about 500 miles of roads during the Third Five-Year Plan.

In waterlogged areas, the use of soil, stabilized with lime, has found successful application as a sub-base or base course for road pavements in the State of Punjab. Encouraged by the initial results longer lengths of roads with similar techniques are being taken up.

It is expected that the decision of the Planning Commission to persuade the States to devote 1 per cent of the cost of the road construction in Third Five-Year Plan to Research Development Projects will give further fillip to the use of local materials in road construction.

2.3.2. Stabilization of Black Cotton Soil

Based on some work carried out in the U.P. P.W.D. Research Institute,[2] Lucknow, on the effect of common salt, alum and white lime on black cotton soils found in Jhansi area, it was indicated that by adding 4 to 5 per cent of common salt or 3 to 4 per cent of white lime (CaO) there was an improvement in the shrinkage characteristics of these soils.

The use of lime for stabilizing black cotton soil was reported by C.R.R.I.[1] wherein it was shown that soil when mixed with lime and compacted, developed sufficient strength; and this fact can be advantageously employed for the purposes of partly replacing stone soling with treated black cotton soil. It is observed that all earth roads in black cotton soil areas are rendered almost impassable during rainy season and, therefore, would require some type of treatment to enable the traffic to pass over them. Experiments have shown that addition of 5 to 10 per cent of calcium hydroxide to the black cotton soil, at as high as 50 to 60 per cent moisture content, would impart enough strength to the soil, to enable light traffic to pass over the road. This strength is retained only as long as water contents are high.

2.3.3. Mechanical Stabilization

Pilot Plant lengths sponsored by the C.R.R.I. using soil, stabilized with soft aggregate in road construction, were completed during last year in the States of Madras and Mysore. The same technique has been extended to States of Orissa, Kerala and Andhra Pradesh. Longer lengths using the same methods are being taken up in the U.P. and Rajasthan.

In Uttar Pradesh[2], experimental stretches were constructed using the following variables for the wearing course to study their relative performance.

A. Soil - Kankar mixes using 35 per cent and 40 per cent of kankar.

B. Soil - Brick mixes using 40 per cent brick aggregates but using first class, overburnt and third class bricks.

C. Wearing courses consisting of wholly kankar, brick ballast, brick on edge and flat brick in place of stabilized soil wearing course.

For base courses the variables introduced were Stabilized soil base course, compacted kankar and flat bricks. The average traffic on this road is about 200 tons and goes upto 400 tons in peak period.

The use of local moorum (called gravel in the States of Madras and Mysore), deposits, complying with the principles of mechanical stabilization has been successfully tried by the Madras Highway Research Station[3]. The exact place of such material in road pavement was determined on the basis of C.B.R. tests.

Under the Pilot Plant Scheme of C.R.R.I., road lengths of about one mile each, using soil-gravel specifications have been constructed in the States of Jammu and Kashmir, Kerala and Orissa. The lengths are reported to be giving satisfactory service.

2.3.4. Chemical Stabilization

Research work on the stabilization of soils with chemicals has been in progress in various research laboratories in India. The use of small concentration of chemicals like Arquad 2HT, Armac T, D.D.B., Cationic Emulsifier and Silicone, was reported earlier by C.R.R.I.[1] and it was observed that Arquad 2HT, Armac T and Silicone gave better results than other chemicals.

In continuation of the work mentioned last year by the College of Military Engineering regarding the improvement in plasticity characteristics of some soil groups by the addition of small concentrations of dispersants like Sodium Iexa-meta phosphate and Sodium-tri-Polyphosphate, it had now been stated[4] that the dispersant treated soils gave higher densities than untreated soils for the same compactive effort. Regarding improvement in strength characteristics of soils pretreated with these dispersants before the admixture of cement, compared to those where cement was directly mixed, it had been said that soils of C L group gave best results. It was also reported that highly clayey soils did not respond as well as those in C L group though their response was better than silty soils, and that the black cotton soils gave very variable results.

College of Military Engineering carried out some work also on the treatment of different types of soils with quaternary ammonium salts. The four types of ammonium salts tried are given below :

(i)	2HT—75 per cent	Arqud Dialkyl Quaternary ammonium chloride.
(ii)	12—50 per cent	Arquad-Alkyl Quaternary ammonium chloride.
(iii)	16—50 per cent	,, ,,
(iv)	18—50 per cent	,, ,,

It was stated that the results obtained, so far show that the salts produce some water-proofness, but no gain in strength.

2.3.5. Cement Stabilization

Stabilization of local sand with 2 per ccent cement in connection with the design of sub-base over a sandy subgrade was tried by Madras[3].

Sand stabilized with 2.0 per cent cement was found to be more economical than sand-moorum and sand-clay mixes in some of the reaches for the particular conditions existing there. West Coast Road Division of the Bombay P.W.D. conducted[5] a field experiment on the West Coast Road, for studying the relative performance of soil-cement and the conventional soling and W.B.M., as bases for concrete pavements when constructed on a high embankment soon after it was raised. It was observed that both the sections have developed cracks. As the report did not contain detailed information on the degree of settlements within the embankment and mode of embankment construction, etc., it was not possible to draw specific conclusions from this experiment. For this particular locality where trap stone is cheap and available, it was found that the cost of soil cement base worked out more than the cost of conventional W.B.M. base course.

Work on the stabilization of soils with cement is also in progress in the Roads and Building Research Institute, Government of West Bengal[6].

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3. BRIDGES AND STRUCTURES

General

It is gratifying to note from the reports recived that more and more interest is being taken by organizations in the country for scientific appraisal of problems connected with highway bridges and structures.

3.1. Superstructures

3.1.1. Investigation of bridges for load-carrying capacity.

The Highways Research Station, Madras, was called upon to ascertain whether Napier Bridge at Madras could take up a gross load of 130 tons; as some packages of heavy equipment ranging in weight from 63 to 98 tons received for the Neyveli Lignite Project at Madras Harbour had to be transported over this bridge on 32-ton tractor-trailers.

The bridge was tested for the pay loads of 63 and 98 tons using the 32-ton tractor-trailer. Deflection observations were taken and strain measurements were also recorded for a limited number of points. Due to the complex nature of the structure, it was not possible to calculate deflection of different members, for verification by actual loading tests. It was considered, however, that from the order of deflections under test loads, and the extent of recovery on removal of the loads, a rough idea could be obtained regarding the behaviour of the structure under load. The strain measurements helped to see whether the stresses developed under test loads are within elastic limits. On the basis of this test, the bridge was considered safe for the load proposed to be carried over the bridge.

3.1.2. Use of Prestressed Slabs and Blocks for Strengthening of Culverts and Small span Bridges

The Highway Research Station, Madras, continued work on the use of prestressed slabs and blocks for strengthening of culverts and small span bridges, by evolving a design for post-tensioned composite beam. As stresses are induced due to differential shrinkage between the old members and the fresh concrete added thereon, a study has been undertaken to devise means to minimise the stresses.

3.1.3. Testing on Wire-grips and Anchor-plates of Gifford-Udall System of Prestressing

The Highways Department of Andhra Pradesh has reported the results of tests carried out on Gifford-Udall wire-grips and ¾ in.

2.3.6. Stabilization with Lime and Molasses

Lime stabilization seems to have gained wide application throughout the country as revealed from the work being conducted by various laboratories. B and R Research Laboratory[7], Chandigarh, Punjab carried out work on the effect of lime on the stabilization of soil (silty clay type) in the presence of increasing percentages of sodium sulphate; the main tests carried out being compressive strength, wetting and drying cycles and C.B.R. It was reported that the dry compressive strengths without lime, with different percentages of sulphate were almost the same but as the percentage of lime increased, the strength showed a tendency to fall; while in the case of wet blocks the results were in the reverse order.

The results of the same laboratory indicate that the C.B.R. value of soil containing different percentages of sodium sulphate (ranging from 0.1 to 1.0 per cent) varied from 6 to 30 with the addition to 2.5 per cent lime compared to a value of about 1 per cent for an untreated soil specimen. It was also indicated that even in the presence of sulphates, the per cent loss in weight, in wetting and drying cycles can be reduced by addition of lime.

Trials with lime for improvement of weak subgrades and swelling soils were also in progress in Road and Building Research Institute, West Bengal[6], U.P. P.W.D. Research Institute[2] and Road Research Wing of Mysore Engineering Research Station[8]. The Railway Research Section of the Railway Research, Designs and Standards Organization, Lucknow[9], had reported that lime stabilization was being tried for some Railway embankments in black cotton soil areas.

As stated last year, work on evolving specifications for light traffic roads in rural areas has shown that when 2.5 per cent lime and 5 per cent molasses on dry weight of soil, are added to stabilized wearing course mixture (stabilized with brick aggregate) the service quality of the crust improved considerably even without any surface treatment. But, after two years of traffic, the surface started showing signs of abrasion.

2.3.7. Bituminous Stabilization

Stabilization of sand with bitumen was reported by the Highway Research Station, Madras[3]. It was stated that the addition of bitumen alone to sand, while increasing its resistance to softening effect of water, does not add appreciably to its strength in the wet State. As mentioned last year, the addition of 2 per cent of cutback with small percentages of additives like cement, lime, etc., appears to raise the stability value of the soil mix.

Addition of cement and flyash or lime and flyash was found to further increase the stability of sand bituminous mixtures.

2.3.8. Study of Subgrade Moisture

Studies on moisture suction characteristics of soils and sands are in progress in C.R.R.I[1]. to determine the effect of density on the suction characteristics of different soils and sands having different fineness moduli. A knowledge of these relationships would be very useful in determining the moisture contents at which different soils will be in equilibrium for particular conditions of water-table prevailing at the site.

2.3.9. Low Grade Aggregates

Preliminary investigations in C.R.R.I[1], on moorum samples from West Bengal[6], have shown that some of the moorums respond very favourably to treatment with lime, thereby making them fit for use in the upper layers of road pavement.

2.3.10. Use of Radio Isotopes for Quality control of Highway Fill construction

The principle that gamma rays are affected by the density of the media through which they traverse has been made use of for determining the dry bulk densities of soil in situ at C.R.R.I. Cobalt 60 was used as a source of Energy. The work using different soils at different moistures is in progress.[1]

Sailent Points for Discussion

1. Relative merits of lime and cement stabilization.
2. Improving the road construction in black cotton soil area.
3. Utility of soil cement base in place of W.B.M. under concrete pavements.

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thick anchor-plates, with a view to ascertain their efficiency before adopting the Gifford-Udall system of prestressing for a bridge to be constructed across river Godavari.

It is reported that 7 mm. dia. HTS wire, when tested using Gifford-Udall grips, gave ultimate strength 5 to 10 per cent less than that obtained by using grips of the standard testing machine, due to biting of serrated wedges into wire during prestressing. As against the $\frac{1}{8}$ in. slippage between the wire and anchor grips suggested by the suppliers, actual slippage of the order of $\frac{1}{4}$ in. was observed for 10,000 lb prestressing force. The $\frac{3}{4}$ in. thick anchorage plates showed appreciable deformation and high stress concentration, going beyond yield value in the middle region, under test.

As a basis of this investigation, it was decided to use 1 in. thick end-plates and to use lower design stresses to allow for reduction in wire-strength due to grip-bits.

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II. TRAFFIC AND TRANSPORTATION

1. GEOMETRICS, ECONOMICS, STATISTICS AND SAFETY

1.1. Geometrics

1.1.1. Design of Road Intersections

Reports of the Central Road Research Institute[1] and the Madras Highway Research Station[2] indicated that work on scientific lines was continued on the design of intersections to enable free flow of traffic on some of the busy intersections in Delhi and Madras respectively. Improvements suggested for some of these intersections include introduction of 4-phase traffic signals, separate lane for right-turning vehicles, widening of the carriageway, etc.

1.1.2. Parking Geometrics

A study has been conducted by C.R.R.I. to find out the parking areas required for different types of vehicles with a view to lay down geometric standards for parking reservations. Suitable layouts for parking of different types of vehicles under varying sizes of areas for suggesting the economic size of parking lots were also studied.

1.2. Economics

1.2.1. Effect of delays at road-rail crossings on the Economics of Highway Transportation

As reported in the last session, work on this aspect is being continued in C.R.R.I.[1]

1.3. Statistics and Safety

1.3.1. Parking Standards

The growing parking problem in various cities of the country is further aggravating the traffic problem. A preliminary study of the parking problem in Delhi as a case study, was carried out by C.R.R.I. during the year under report. Various causes of the problem as indicated by the study are (i) rapid expansion of vehicular traffic, (ii) no planned terminal facilities for passenger buses and trucks, (iii) lack of loading and unloading facilities, and (iv) increasing number of persons using private vehicles as a means of transportation. The study indicates two approaches to this problem *i.e.*, (i) controlling further aggravation of the problem, and (ii) meeting the problem as it exists at present. Towards controlling the problem from getting further aggravated, it has been recommended that every new structure or substantially reconstructed structure must have provision for sufficient parking area, for the traffic it is likely to draw. Tentatively, it has been recommended that for cinema houses, there should be 250 sq. ft. of parking area for every 2 seats in the class of Rs 2 per seat and above, and the same area as parking reservation for every 4 seats in the class below Rs 2 per seat. For restaurants and night clubs, a reservation of 250 sq. ft. should be made for every 2 to 3 seats and for stadiums, a reservation of 250 sq. ft. for every 4 seats is recommended. In residential flats, especially multi-storeyed buildings for an income group of Rs 1,000 per month and above 250 sq. ft. must be reserved for every family unit. Towards solving the already existing problem, it has been recommended that restricted and judicious use of the kerb spaces available for parking should be made and underground and multi-storyed garages should not be constructed until the level ground area for parking has been fully made use of. For time restriction on kerb parking, use of parking meters has been recommended. Reservation of loading and unloading zones and time restriction on their use have also been included in the recommendations.

1.3.2. Parking Survey at Mount Road, Madras

A parking survey was conducted on Mount Road, Madras, with a view to prepare an inventory of available parking spaces. The parking needs of different types of vehicles including cyclists were assessed. This survey formed a part of the general improvement scheme for Mount Road. Recommendations made included thirty-minute kerb parking restriction in Mount Road to induce business establishments to provide private parking facilities for their employees, demarcation of parking stalls, providing regulating signs, etc.

1.3.3. Drivers Study

A study regarding drivers' examination, licensing and checking drivers' performance for improvement purposes was carried out by the C.R.R.I. The recommendations included driving tests, psycho-physical and psychological tests, and an introduction of check on drivers' performance by 'Point System'. Under this system, different traffic offences will be assigned different point values so that when a driver is convicted of a particular offence, points for that offence are added on his permanent record. When the added points reach a particular limit, action for his improvement is taken by contacting the driver.

This study has now been extended for establishing methods for improved training, examination and control of drivers. It is

proposed to approach transport operators in various parts of India for collecting information on existing methods for driver training, examination and control. A questionnaire is being prepared for this purpose. It is also proposed to investigate systematically the methods of testing drivers in Delhi with the co-operation of the Delhi State Motor Transport Controller.

1.3.4. Speed and Delay Study

This study conducted by C.R.R.I.[1] aims at locating various spots on congested routes in Delhi, where traffic gets most frequently obstructed, and at assessing the causes of such obstructions and the resultant wastage of time. Traffic surveys of the selected roads are in progress by "Floating Car" method which gives the average travel speed. Speed surveys on certain roads in Madras were conducted by the Highway Research Station[2], Madras, by making spot speed studies using enoscope and based on the 85 percentile speed, the permissible speed limits were arrived at and compared with the legal limits.

1.3.5. Pedestrian Traffic Study

Pedestrians in urban areas are susceptible to maximum danger from traffic accidents. A study made by C.R.R.I.[1] on road accidents in Delhi for the past few years has shown that pedestrian casualties are increasing. This study which aims at pointing out the lack of facilities for pedestrians and the enforcement difficulties concerning traffic rules, has been completed. The recommendations include the provision of side-walks, proper signal phasing, railings, cross-walk markings at intersections, flashing signals and various other pedestrian safety devices.

A flashing signal for pedestrians' safety along with a newly designed CROSS-WALK sign was installed on an important road in New Delhi with a view to study its effect. The results obtained are now under study.

1.3.6. Study of Cycle Traffic

The different aspects of the problem of cycle traffic in Delhi and New Delhi are being studied by the C.R.R.I. for the last few years[3]. Some aspects which have since been studied and reported earlier are: cycle traffic surveys and their analysis, suitable grades and lengths conveniently negotiable by cyclists, geometric design requirements of cycle tracks, provision of express cycleways, etc.

Results from further investigations on a few other aspects of the problem, conducted this year, have indicated the necessity of pursuing a two-pronged approach towards tackling the cycle traffic problem, namely, (a) exercising control on cycle traffic to improve the cyclists' behaviour for making the maximum possible use of the facilities available at any time, and (b) making every effort for providing adequate facilities for cycle traffic. On this basis, some major recommendations have been made.

1. Cycles should be registered like motor vehicles.
2. Every cyclist should be required to hold a riding licence. This is meant to improve the behaviour of cyclist on the road, as well as to render enforcement of the road rules an easier task.
3. Parking facilities for cyclists must be provided around all new constructions wherever needed, and also in the existing areas where these facilities are inadequate at present.
4. Some safety features should be added to the cycle at the manufacturing stage such as built in lamps, reduction in cycle height, luminiscient strips on cycle pedals, etc.
5. For purposes of future planning, the one-way distance to be travelled by cyclists should not be kept more thon 4 to 5 miles at the maximum.

1.3.7. Roads Accidents Study

(a) A study to investigate into the causes of road accidents and suggest preventive measure has been taken up by the C.R.R.I.[1] Road accident statistics for Delhi pertaining to 1958 have been analysed by the Hollerith Punch-Card system. The results will be published separately. The accident data for 1959-60 are now under investigation.

(b) Road accident records for the City of Madras are being analysed by the Highway Research Station,[2] Madras. Some preliminary analysis for the past years from 1956 to 1959 has shown that cyclists were the worst sufferers amongst those using slow-moving vehicles.

Salient Points

1. Functions of 4 phase signals in traffic control.
2. Drivers testing and training—its importance in road safety.

3. Uses of road accident analysis for designing preventive measures.
4. Parking designs and standards.
5. Controls and facilities for cycle traffic.
6. Controls and facilities for pedestrian traffic.

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INFORMATION SECTION

"SOLUTION OF A RECTANGULAR PLATE SUPPORTED AT THE CENTRE AND SUBJECTED TO UNIFORM LOADING"

By

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1. INTRODUCTION

Upward warping of the edges of concrete slabs, when the top surface is at a lower temperature than the bottom surface, as during night, is a well-known phenomenon. Under these circumstances, the edges lose subgrade support and become most vulnerable under wheel load. The aim of this Paper is to determine, for a slab of known width and thickness, the economical spacing of the contraction joints, such that the self weight of the slab is sufficient to neutralise the warping effect.

The problem has two parts :

(i) determination of amount of the upward deflection of the edges of a weightless slab on account of temperature gradient ; and

(ii) determination of the amount of downward deflection of the edges of the slab on account of its own weight considering that the slab is supported at the centre.

The present Paper attempts to solve only the second part of the problem, and further work on it is in progress. The solution, however, has a general application and can be utilised for finding out, say, the stresses in mushroom slabs, shown in Fig. 1 (Plate I).

2. THE EXACT METHOD

The general differential equation for a rectangular plate subjected to any form of loading, as developed by Sophie Germain[1], is :

$$D \nabla^2 \nabla^2 \bar{w} = p \qquad \dots (1)$$

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where

$$D = \frac{E\,(2h)^3}{12-(1-\mu^2)} = \text{stiffness of the plate}$$

E = modulus of elasticity

μ = Poisson's ratio

$2h$ = thickness of the plate

$\bar{w}$ = deflection of a point on the plate normal to its surface

and p = load per unit area.

The exact solution of this equation under various limiting conditions involves laborious work and in many cases may be very difficult. For practical purposes, however, approximate methods may be used. For solving this particular problem, the approximate Strain Energy Method after Ritz[2] has been followed.

STRAIN ENERGY METHOD

The total energy of a deformed plate consists of two parts :

(1) energy on account of the deflection due to the superimposed load ;

(2) potential energy.

The energy on account of the deflection $= \iint p\,\bar{w}\,(x, y)\, dx\, dy$.

The potential energy, obtained by summation of the energies due to normal and shear forces, is given by :

$$V = \frac{D}{2}\iint\left[\left(\frac{\partial^2 w}{\partial x^2}\right)^2 + \left(\frac{\partial^2 w}{\partial y^2}\right)^2 + 2\,\mu.\,\frac{\partial^2 w}{\partial x^2}\,.\,\frac{\partial^2 w}{\partial y^2} + 2\,(1-\mu)\left(\frac{\partial^2 \bar{w}}{\partial x\,\partial y}\right)^2\right] dx\, dy \qquad \dots (2)$$

The total strain energy U, therefore is given by :

$$U = -\left[\iint p\bar{w}\,(x, y)\, dx\, dy + V\right] \qquad \dots (3)$$

For satisfactory use of the Strain Energy Method, function of deformation of the elastic surface should be such that all the geometrical end conditions are fulfilled and that the deformation corresponds to the given loading. The approximate solution will be obtained, if certain arbitrary parameters $\beta_1, \beta_2, \beta_3, \dots\dots\beta_n$, whose values must be such as to give an extreme value to the total energy of system including the load, are introduced in the expression for deformation. Thus if the deformation $(\bar{w})$ is expressed by the trigonometrical polyonym :

$$\bar{w} = \beta_1\left(1-\cos\frac{\pi x}{a}\right) + \beta_2\left(1-\cos\frac{\pi y}{b}\right) + \beta_3\left(1-\cos\frac{\pi x}{a}\right) + \beta_4\left(1-\cos\frac{\pi y}{b}\right) + \dots\ \beta_{n-1}\left(1-\cos\frac{\pi x}{a}\right) + \beta_n\left(1-\cos\frac{\pi y}{b}\right) \quad \dots (4)$$

where a and b are the length and the breadth of the plate respectively, then for the extreme value of the total energy (U) of the system, the following must hold true :

$$\frac{\partial U}{\partial \beta_1} = 0, \quad \frac{\partial U}{\partial \beta_2} = 0 \ \dots\dots\dots\dots \ \frac{\partial U}{\partial \beta_n} = 0.$$

3. SOLUTION

3.1. End Conditions

For small deformations, the expression in equation (4) may be approximated as :

$$\bar{w} = \beta_1\left(1 - \cos\frac{\pi x}{a}\right) + \beta_2\left(1 - \cos\frac{\pi y}{b}\right) \qquad \dots (4a)$$

The expression in equation (4a) fulfils the conditions that the deformation at the centre would be zero and at the corners maximum.

Thus,

$$\bar{w}\,(x, y) = 0 \quad \text{when } x = o,\ y = o$$

$$= \beta_1 \quad \text{when } x = \pm\frac{a}{2},\ y = o$$

$$= \beta_2 \quad \text{when } x = o, y = \pm \frac{b}{2} \text{ and}$$

$$= (\beta_1 + \beta_2) \text{ (maximum) when } x = \pm \frac{a}{2}, y = \pm \frac{b}{2}.$$

See also Table 1.

But two more statical end conditions have to be fulfilled. Referring to Fig. 3 (Plate I),

(i) For $x = \pm a/2$, the moment about Y-axis (M_y) must be zero

i.e., $$M_y = -D\left(\frac{\partial^2 \bar{w}}{\partial x^2} + \mu \frac{\partial^2 \bar{w}}{\partial y^2}\right) = 0$$

(ii) For $y = \pm b/2$, M_x must be zero

i.e., $$M_x = -D\left(\frac{\partial^2 \bar{w}}{\partial y^2} + \mu \frac{\partial^2 \bar{w}}{\partial x^2}\right) = 0$$

Putting the values of $\bar{w}(x, y)$ from equation (4a) in M_y and M_x under the above conditions, we get

$$\left.\begin{aligned} M_y &= -D\left[\beta_1 \frac{\pi^2}{a^2} \cos\frac{\pi x}{a} + \mu \beta_2 \frac{\pi^2}{b^2} \cdot \cos\frac{\pi y}{b}\right] \\ M_x &= -D\left[\beta_2 \frac{\pi^2}{b^2} \cos\frac{\pi y}{b} + \mu \beta_1 \frac{\pi^2}{a^2} \cdot \cos\frac{\pi x}{a}\right] \end{aligned}\right\} \quad \ldots (5)$$

The expressions in equation (5) consist of two parts, the major parts of which, when $x = \pm a/2$ and $y = \pm b/2$ for M_y and M_x respectively, would be zero, while the minor parts containing μ are not 0 in general. As a result, the statical end conditions of the plate are not fully satisfied. However, the error arising out of this, for all practical purposes, could be neglected.

3.2. Energy Equation

For the whole plate, the energy equation (2) can be written as

$$V = 2D \int_0^{a/2} \int_0^{b/2} \left[\left(\frac{\partial^2 \bar{w}}{\partial x^2}\right)^2 + \left(\frac{\partial^2 \bar{w}}{\partial y^2}\right) + 2\mu \frac{\partial^2 \bar{w}}{\partial x^2} \cdot \frac{\partial^2 w}{\partial y^2} + 2(1-\mu)\left(\frac{\partial^2 w}{\partial x \, \partial y}\right)^2\right] dx\, dy$$

Now by substituting the value of $\bar{w}$ from equation (4a)

$$\int_0^{a/2} \int_0^{b/2} \left(\frac{\partial^2 w}{\partial x^2}\right)^2 dx dy$$

$$= \int_0^{a/2} \int_0^{b/2} \left[\frac{\partial^2}{\partial x^2}\left\{\beta_1\left(1 - \cos\frac{\pi x}{a}\right) + \beta_2\left(1 - \cos\frac{\pi y}{b}\right)\right\}\right]^2 dx\, dy$$

$$= \beta_1^2 \cdot \frac{\pi^4}{a^4} \cdot \frac{ab}{8},$$

Similarly,

$$\int_0^{a/2} \int_0^{b/2} \left(\frac{\partial^2 \bar{w}}{\partial y^2}\right)^2 dx\, dy$$

$$= \beta_2^2 \cdot \frac{\pi^4}{b^4} \cdot \frac{ab}{8},$$

$$\int_0^{a/2} \int_0^{b/2} 2\mu \frac{\partial^2 w}{\partial x^2} \cdot \frac{\partial^2 w}{\partial y^2} \cdot dx\, dy$$

$$= 2\mu \int_0^{a/2} \int_0^{b/2} \frac{\partial^2}{\partial x^2}\left[\beta_1\left(1 - \cos\frac{\pi x}{a}\right) + \beta^2\left(1 - \cos\frac{\pi y}{b}\right)\right]$$

$$\cdot \frac{\partial^2}{\partial y^2}\left[\beta_1\left(1 - \cos\frac{\pi x}{a}\right) + \beta_2\left(1 - \cos\frac{\pi y}{b}\right)\right] dx\, dy$$

$$= 2\mu\, \beta_1\, \beta_2 \frac{\pi^2}{ab},$$

and $$\int_0^{a/2} \int_0^{b/2} \left[2(1-\mu)\left(\frac{\partial^2 w}{\partial x\, \partial y}\right)^2\right] dx\, dy$$

$$= 2(1-\mu) \int_0^{a/2} \int_0^{b/2} \left[\frac{\partial^2}{\partial x\, dy}\left\{\beta_1\left(1 - \cos\frac{\pi x}{a}\right) + \beta_2\left(1 - \cos\frac{\pi y}{b}\right)\right\}\right]^2 dx\, dy$$

$$= 0$$

$$\therefore \quad V = 2D\left[\beta_1^2 \frac{\pi^4}{a^4} \cdot \frac{ab}{8} + \beta_2^2 \cdot \frac{\pi^4}{b^4} \cdot \frac{ab}{8} + 2\mu\, \beta_1\, \beta_2 \cdot \frac{\pi^2}{ab}\right]$$

Similarl, by substituting for $\bar{w}$ in the first part of eqn. (3):

$$\int\int p\bar{w}(x, y)\ dx\ dy = 4\int_0^{a/2}\int_0^{b/2} p\left[(\beta_1 + \beta_2) - \left(\beta_1 \cos\frac{\pi x}{a} + \beta_2 \cos\frac{\pi y}{b}\right)\right] dx\ dy$$

$$= 4p\int_0^{a/2}\left[(\beta_1 + \beta_2)\frac{b}{2} - \left(\beta_1 \cos\frac{\pi x}{a} \cdot \frac{b}{2} + \beta_2 \cdot \frac{b}{\pi}\right)\right] dx$$

$$= 4p\left[(\beta_1 + \beta_2)\frac{b}{2} \cdot \frac{a}{2} - \left(\frac{b}{2} \cdot \beta_1 \cdot \frac{a}{\pi} + \beta_2 \cdot \frac{b}{\pi} \cdot \frac{a}{2}\right)\right]$$

$$= 4p\left[ab\,(\beta_1 + \beta_2)\left(\frac{1}{4} - \frac{1}{2\pi}\right)\right]$$

$$= p\,ab\,(\beta_1 + \beta_2)\left(1 - \frac{2}{\pi}\right)$$

Hence equation (3) can be written as:

$$U = -\left[pab\,(\beta_1 + \beta_2)\left(1 - \frac{2}{\pi}\right) + 2D\left\{\beta_1^2 \cdot \frac{\pi^4}{a^4} \cdot \frac{ab}{8} + \beta_2^2 \cdot \frac{\pi^4}{b^4} \cdot \frac{ab}{8} + 2\mu\, \beta_1\, \beta_2 \cdot \frac{\pi^2}{ab}\right\}\right] \quad \ldots \quad (6)$$

Now for the extreme value (maxima or minima) of the total energy U of the system, the following must hold true:

(a) $\dfrac{\partial U}{\partial \beta_1} = 0$

$$\text{i.e.,} \quad -\left[pab\left(1 - \frac{2}{\pi}\right) + 2D\left[\frac{\pi^4}{a^4} \cdot \frac{ab}{8} \cdot 2\,\beta_1 + 2\mu\,\beta_2\,\frac{\pi^2}{ab}\right]\right] = 0 \quad \ldots \quad (7)$$

and

(b) $\dfrac{\partial U}{\partial \beta_1} = 0$

$$\text{i.e.,} \quad -\left[pab\left(1 - \frac{2}{\pi}\right) + 2D\left[\frac{\pi^4}{b^4} \cdot \frac{ab}{8} \cdot 2\beta_2 + 2\mu\beta_1 \cdot \frac{\pi^2}{ab}\right]\right] = 0 \quad \ldots \quad (8)$$

Solving the equations (7) and (8) for β_1 and β_2 we get

$$\beta_1 = \Omega \cdot a^4\left(\pi^2 - 8\mu\frac{b^2}{a^2}\right) \cdot p \quad \ldots \quad (9)$$

$$\beta_2 = \Omega \cdot b^4\left(\pi^2 - 8\mu\frac{a^2}{b^2}\right) \cdot p \quad \ldots \quad (10)$$

where Ω is a factor depending upon D and μ and

$$\Omega = \frac{2\pi - 4}{D.\ \pi^3.\ (64\mu^2 - \pi^4)}$$

$$= \frac{0.88\,(1 - \mu^2)}{(64\mu^2 - \pi^4).\ E.\ (2h)^3}$$

3.3. Bending Moments and Deflections

Let us consider a few critical cases, for which bending moments (M_y and M_x) and deflections are found out.

Rewriting eqn. (5),

$$M_y = -D\left[\beta_1 \cdot \frac{\pi^2}{a^2} \cdot \cos\frac{\pi x}{a} + \mu\,\beta_2 \cdot \frac{\pi^2}{b^2} \cdot \cos\frac{\pi y}{b}\right]$$

$$M_x = -D\left[\beta_2 \cdot \frac{\pi^2}{b^2} \cdot \cos\frac{\pi y}{b} + \mu\,\beta_1 \cdot \frac{\pi^2}{a^2} \cdot \cos\frac{\pi x}{a}\right].$$

Case A: $x = y = 0$, i.e., centre point (I) of the plate, in Fig. 3. (Plate I).

(a) $M_y = -D\left[\beta_1 \cdot \dfrac{\pi^2}{a^2} + \mu\,\beta_2 \cdot \dfrac{\pi^2}{b^2}\right]$.

(b) $M_x = -D\left[\beta_2 \cdot \frac{\pi^2}{b^2} + \mu\, \beta_1 \cdot \frac{\pi^2}{a^2}\right]$.

and from eqn. 4 (a)

(c) $w(x, y) = 0$

Case B : $x = \pm a/2$, $y = \pm b/2$, i.e., corner points (II) of the plate.

(a) $M_y = 0$

(b) $M_x = 0$

(c) $\bar{w}(x, y) = \beta_1 + \beta_2$

Case C : $x = \pm a/2$, $y = 0$, i.e., mid point (III) on the shorter edge of the plate.

(a) $M_y = 0$

(b) $M_x = -D\left[\beta_2 \cdot \frac{\pi^2}{b^2}\right]$

(c) $w\ (x, y) = \beta_1$

Case D : $x = o$, $y = \pm b/2$, i.e., mid point (IV) on the longer edge of the plate.

(a) $M_y = -D\left[\beta_1 \cdot \frac{\pi^2}{a^2}\right]$

(b) $M_x = 0$

(c) $\bar{w}\ (x, y) = \beta_2$

Once bending moments are obtained, the stresses (σ) can be very easily calculated from the simple Bending Theory.

$\sigma = \frac{M}{Z}$, where Z is the section modulus.

Therefore, $\sigma y = \frac{3M_y}{2h^3}$

and $\sigma\ x = \frac{3M_x}{2h^3}$, $2h$ being the thickness of the plate.

4. TABLES

Values of Ω, β_1 and β_2 are displayed in Tables 2 and 3 and Fig. 4 and 5 (Plate 1) for the following conditions (predominantly suit-

concrete road conditions) :

(a) Material: Concrete

(b) $E = 3 \times 10^5\ kg/cm^2$

(c) $\mu = 0.2$

(d) $b = 3m$

TABLE 1

Values of deflections $\bar{w}\ (x, y)$

x	y	$\bar{w}(x, y)$	Remarks
0	0	0	
$\pm a/2$	0	β_1	a and b length and breadth of the plate respectively
0	$\pm b/2$	β_2	
$\pm a/2$	$\pm b/2$	$\beta_1+\beta_2$	

TABLE 2

Values of Ω

$2h$ (cm)	μ	E (kg/cm²)	Ω (kg⁻¹cm⁻¹)
10	0.2	3×10^5	-0.296×10^{-10}
20	0.2	3×10^5	-0.037×10^{-10}
30	0.2	3×10^5	-0.011×10^{-10}

TABLE 3

Values of β_1 and β_2

[See Figs. 4 and 5 (Plate I) also]

(μ = 0.2, $E = 3 \times 10^5$ kg/cm² and q = self wt. of the plate/cm²)

2 h (cm)	a (m)	a/b	β_1	β_2	Remarks
10	3	1	$-2.00q$	$-2.00q$	
	4	1.33	$-6.88q$	$-1.66q$	
	5	1.67	$-17.3q$	$-1.30q$	
	6	2.00	$-36.5q$	$-0.82q$	
20	3	1	$-0.25q$	$-0.25q$	
	4	1.33	$-0.86q$	$-0.21q$	Negative sign means downward deflection
	5	1.67	$-2.16q$	$-0.16q$	
	6	2.00	$-4.56q$	$-0.10q$	
30	3	1	$-0.08q$	$-0.08q$	
	4	1.33	$-0.25q$	$-0.06q$	
	5	1.67	$-0.64q$	$-0.05q$	
	6	2.00	$-1.35q$	$-0.03q$	

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PLATE I

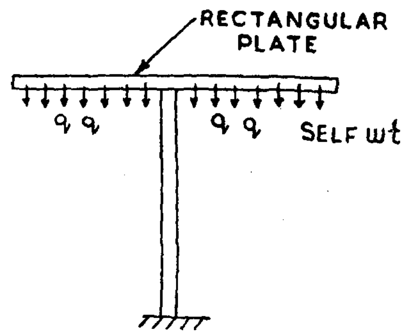

FIG. 1.

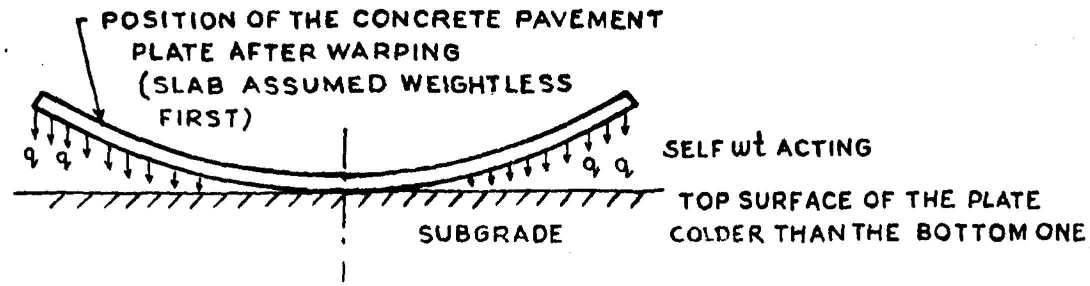

FIG. 2.

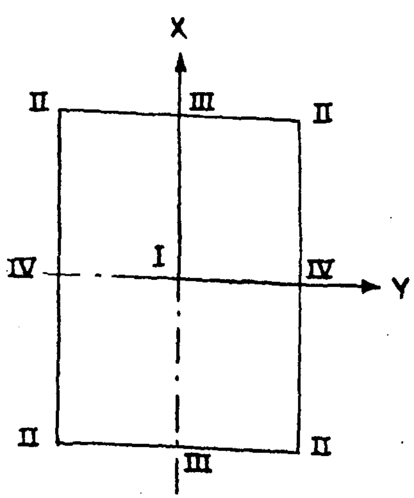

FIG. 3.

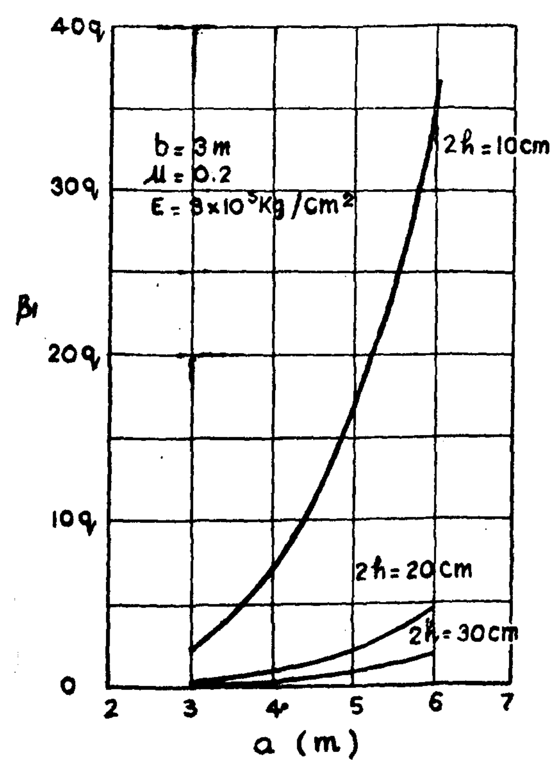

FIG. 4.

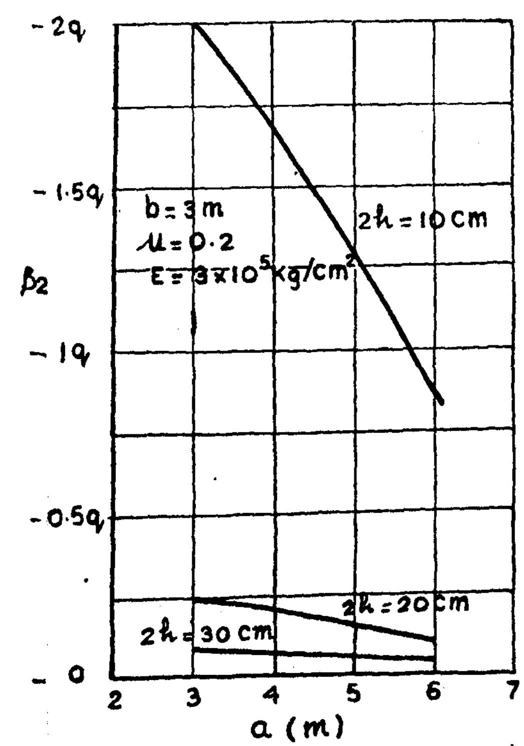

FIG. 5.