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MADRAS HIGHWAYS

THE JOURNAL

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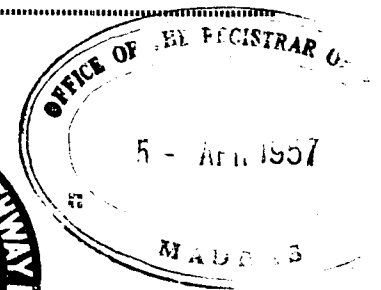
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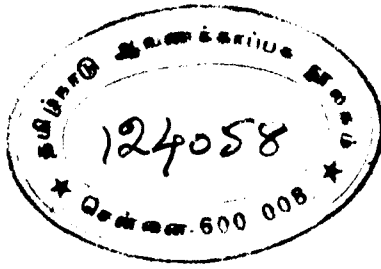
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from the
EDITOR



*The Views expressed by the authors of
the articles are purely their individual
Views and not those of association.*

Railway Tragedy Near Ariyalur

The terrible railway disaster near Ariyalur which occurred in the early hours on 22—11—1956 is perhaps the biggest railway accident which occurred in South India in our living memory. In the tragedy was involved the death of 147 human lives. The lives lost cannot be regained and the bereavement suffered by many families cannot be lightened by any amends. While our heartfelt sympathies go to the victims of this ghastly disaster, there must be a true heart searching on the part of every engineer.

Whatever may be the cause of such a calamity, it must be found out and eliminated and the lives of our fellowmen who put in every day so much trust in us, Engineers, be it when they travel on road, rail, air or water must be safe guarded. It is one thing when a calamity occurs by the act of God as the havoc caused by earthquakes etc. and it is quite another when they are the results of the inefficiency and negligence on the part of the engineers who are responsible for the design and up keep of public works like buildings, bridges and dams.

The attention of the public is much focussed on a railway tragedy of the type that occurred near Mahbubnagar or Ariyalur as they took a huge toll of precious human lives and often the public demand an enquiry into the causes of accident and in this a lot of publicity is given. In the case of road accidents which

more often involve only a few passengers in cars or buses, the tragedy goes unnoticed because the unit carrying passengers is small. But let us remember that the problem of safe guarding the lives of our fellowmen is no less important on road than on rail, causes being similar.

Unlike other forms of engineering design which involve an appreciation and application of the laws of inanimate nature, highway design involves the element of human behaviour in addition and is therefore not entirely susceptible to precise forms of design based on an observance of the natural law. Accidents occur primarily as a result of a lack of timely appreciation of a situation and is therefore of the essence of good design that it reduces the number of occasions on which unexpected or complex situations arise. Design thus becomes a blend of applied scientific principles and an analysis of human behaviour based on observation and accident investigation.

Hence let us be aware that we, Highway Engineers have a special responsibility of protecting the lives of the public when they put so much trust in us by making use of the road and bridges designed constructed and maintained by us. Every one of us, whether he is working in the design of roads and bridges or constructing them at worksite or inspecting the structure in service must have a keen sense of moral responsibility for protecting the lives of the public. Let us be worthy of the faith that our fellowmen have in us.

Submersible Bridge Across River Cooum

at 27/5 of Tirumushi—Satyavedu Road

The pictures adjoining show the details of construction of the submersible bridge across River Cooum at 27/5 of Tirumushi Satyavedu Road which was opened to traffic by Sri M. Bhaktavatsalam, Minister for Agriculture, Madras on 28—11—1956.

Tirumishi - Satyavedu Road is a major district road and is the only route to Chingleput, Conjeevaram and other places south from Tiruvellore and its neighbourhood. The traffic on this road is very heavy, about 1,500 tons per day, and the existing causeway was not able to meet the requirements. It was therefore felt imperative to improve it to suit the present day traffic. But there were other points to be considered before the improvement of the causeway could be taken up. The existing causeway is located very close to the Tiruvellore Railway station and there is a very busy level crossing in the approach to the causeway, practically inside the station yard within its shunting range. Tiruvellore is an important station in the heavily worked Madras - Arkonam railway line and the level crossing gates are closed more than 100 times a day, each closure ranging from 5 minutes to 45 minutes. It can therefore be seen that the level crossing was a worse bottleneck than the causeway.

It was therefore thought not worthwhile to improve the causeway and it was decided to reconstruct it at a site about $1\frac{1}{2}$ furlongs upstream outside shunting limits. Though this involved a new level crossing in its approach, the closure of the gates was not expected to be so numerous here since it was located beyond shunting range. As river Cooum is a regulated one it was

thought that a submersible bridge would serve the purpose. Hence it was that a submersible bridge was taken up for construction at an estimated cost of 2.86 lakhs.

Borings taken at the site revealed that sand existed to a depth of more than 16 ft. sandy loam underlying. A multi-cell box culvert appeared to be the ideal structure for the site conditions and hence it was proposed to construct 8 units of multi-cell box culvert consisting of 4 cells of 10' x 10' to give a lineal waterway of 320'. The boxes rest on reinforced cement concrete caissons 6'-0" deep with cut off walls 6" thick cast monolithically with the members of the boxes. Semi circular cut waters are provided for deck slab and vertical members. The bridge is designed to carry I.R.C. standard class A vehicles for 2 lanes. The clear roadway over the bridge is 22'-0". The design feature of this rigid type of structure will be of interest and it will follow in the next issue of the journal.

Details of construction are given in the drawings accompanying.

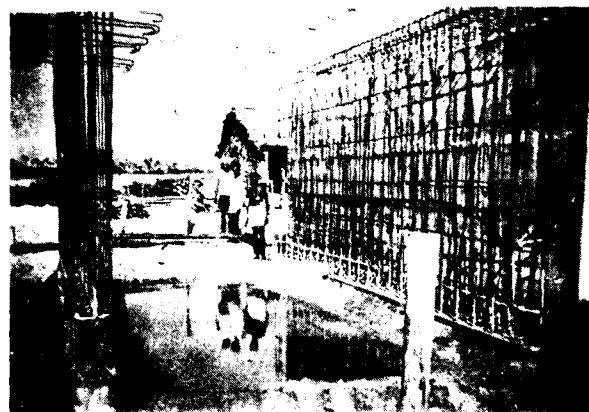
In order to provide good bank connection R. C. counterfort type splayed wings have been provided over R. C. caissons. Details of construction is given in the drawings appended.

Aprons of rough stone dry packing 1'-0" thick over gravel base 6" have been laid to a width of 8'-0" on the Upstream and 12'-0" on the downstream side as a protection.

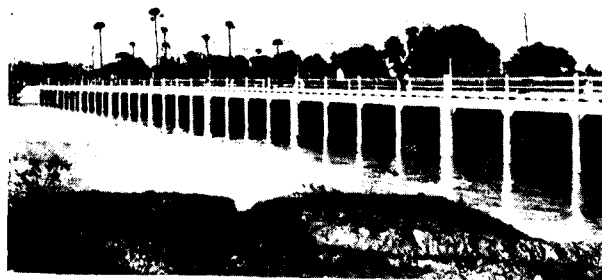
About 405 tons of cement and 76 tons of steel have gone into the work.



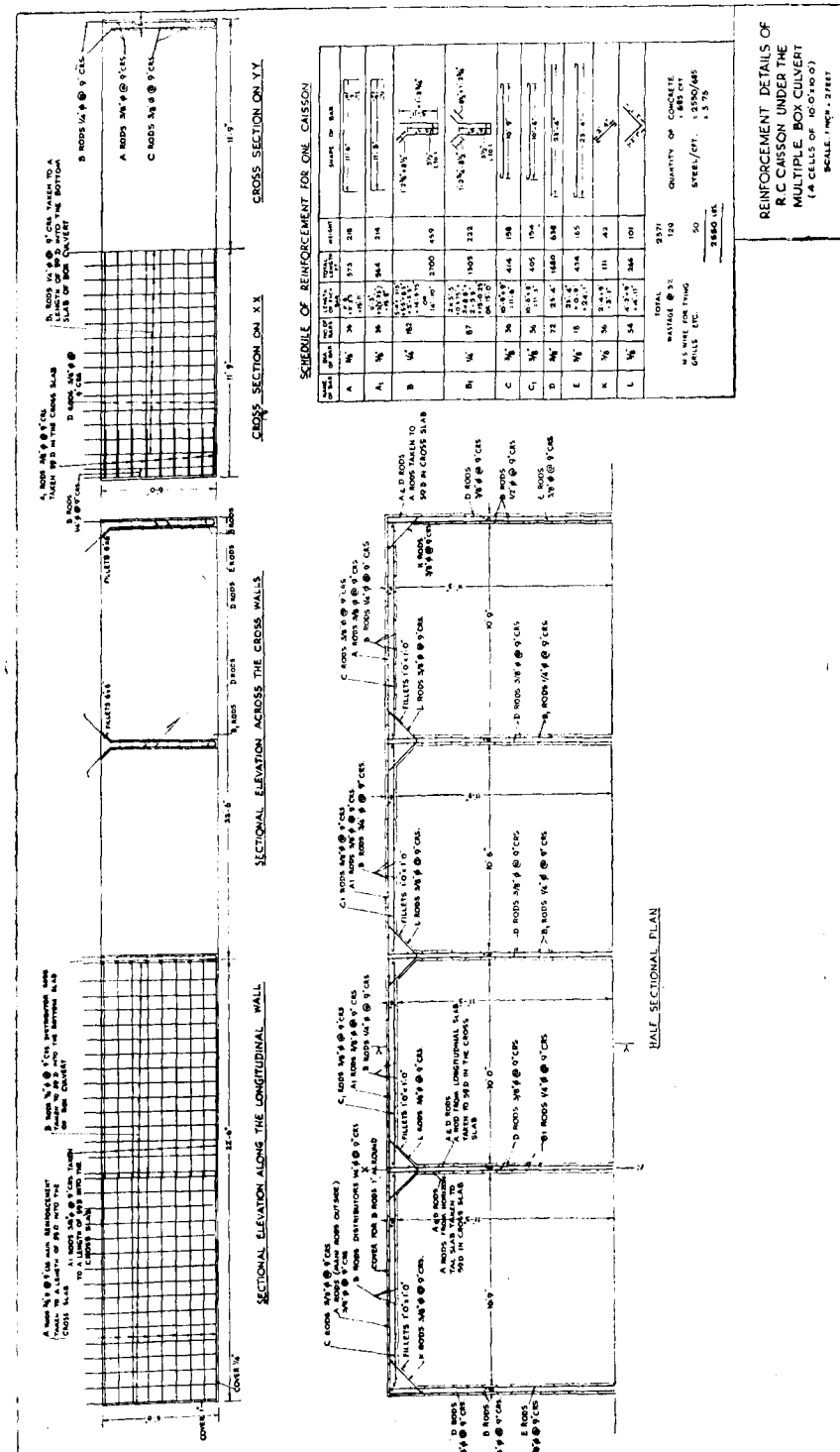
The Submersible bridge under construction

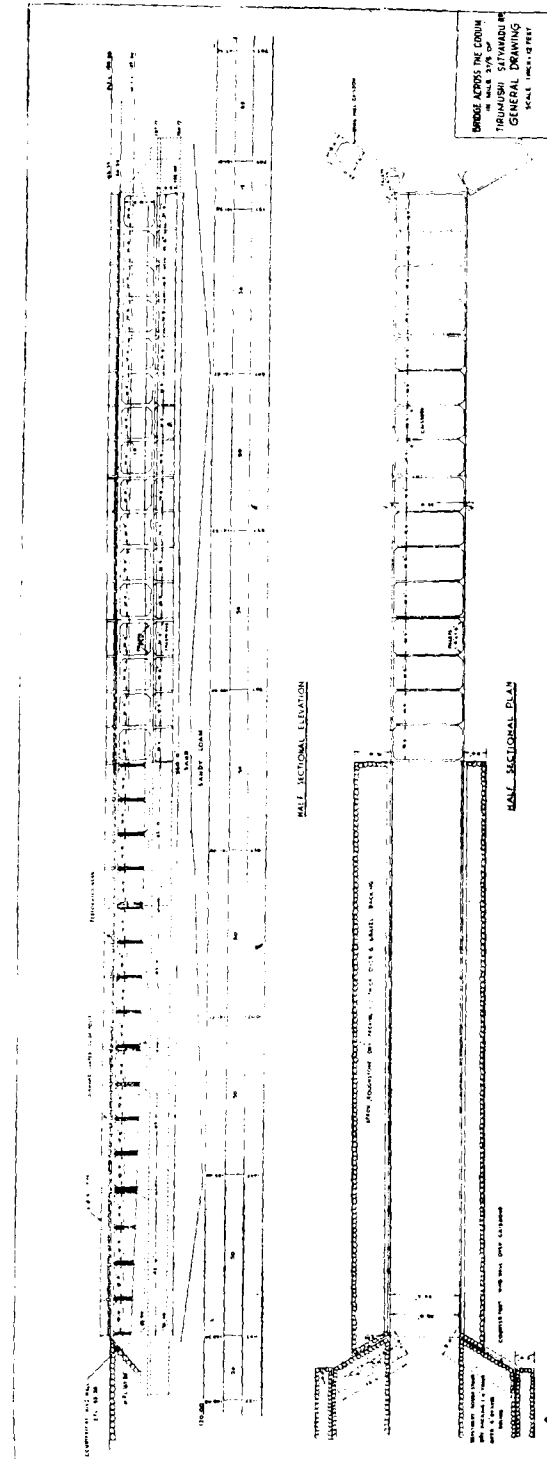
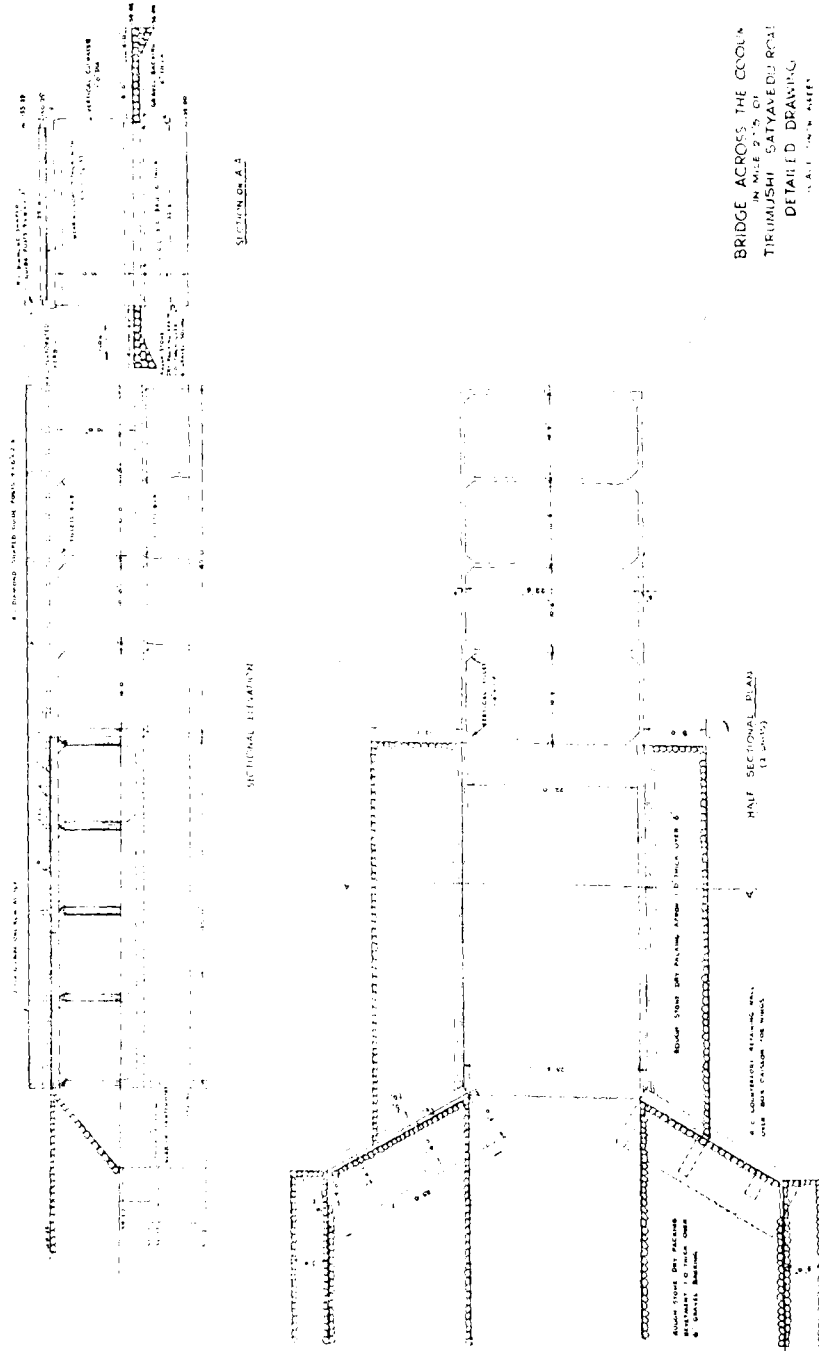


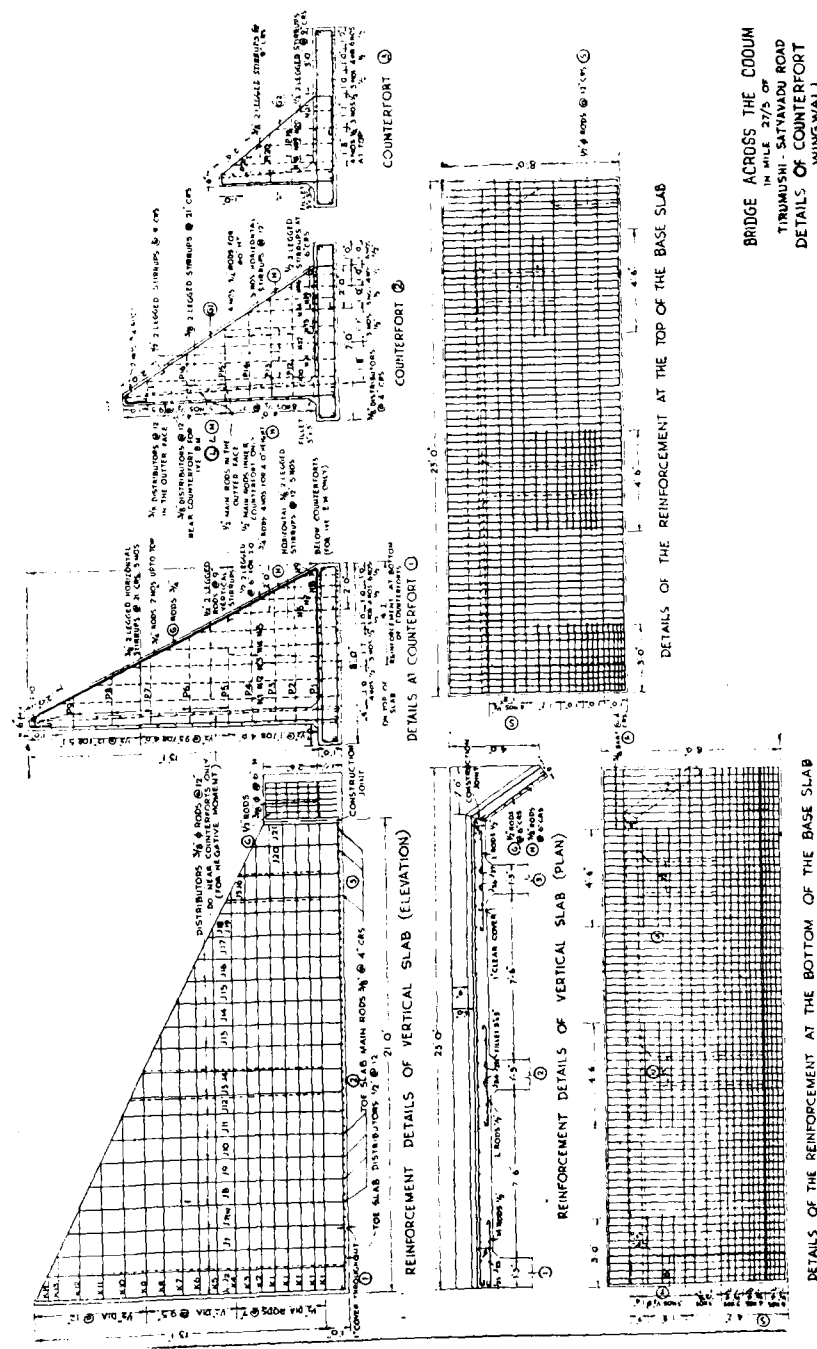
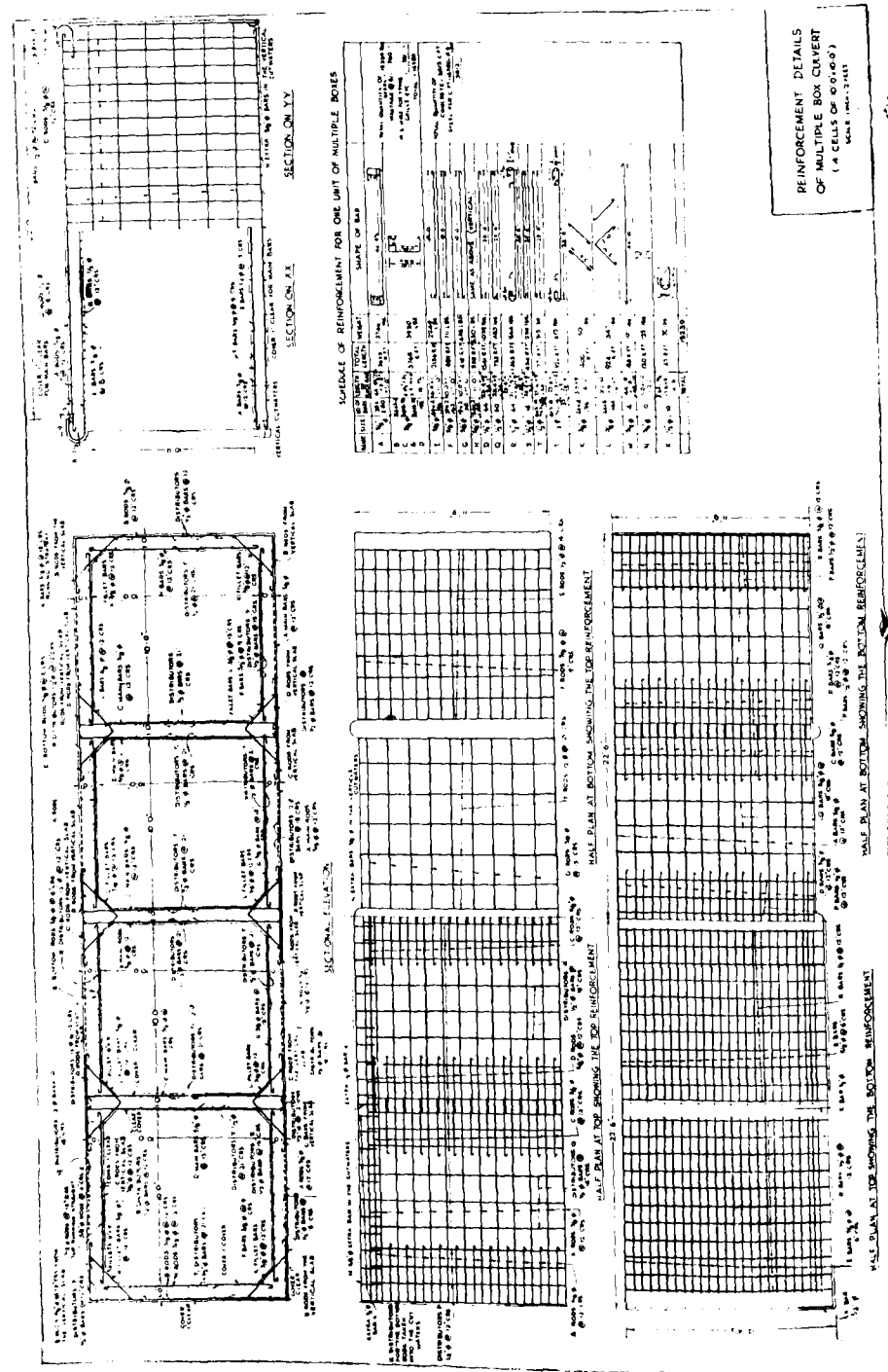
A view of the construction



A view of the completed Submersible bridge







BRIDGE ACROSS THE COOUM
IN MILE 27/5 OF
TIRUMUSHI - SATYAVADU ROAD
DETAILS OF COUNTERFORT
WING WALL

BOTTOM SLAB									
NAME	SIZE	A	B	C	D	E	F	G	H
R	3/8"	7'-0"	8'-0"	10'-195	18'-1074	27'-248	34'-556	40'-576	54'-610
S	1/2"	3'-0"	3'-0"	3'-0"	3'-0"	3'-0"	3'-0"	3'-0"	3'-0"
T	1/2"	3'-0"	3'-0"	3'-0"	3'-0"	3'-0"	3'-0"	3'-0"	3'-0"
U	1/2"	3'-0"	3'-0"	3'-0"	3'-0"	3'-0"	3'-0"	3'-0"	3'-0"

SHAPE OF BAR	

WEIGHT OF RODS... 2195 LBS
WASTAGE... 11%
BINDING WIRE... 25
2320 LBS

TOTAL QUANTITY OF
CONCRETE... 381 CFT.
STEEL PER CFT... 5.09 LBS

BRIDGE ACROSS THE COOUM
IN MILE 27 1/2 OF
TIRUMUSHI - SATYAVEDU ROAD
SCHEDULE OF REINFORCEMENT
FOR COUNTERFORT WINGS
NOT TO SCALE

SCHEDULE OF REINFORCEMENT FOR COUNTERFORT									
NAME	SIZE	A	B	C	D	E	F	G	H
G	3/8"	14'-6"	10'-0"	2'-0"	6'-0"	5'-0"	1'-0"	36'-0"	18'-0"
H	3/8"	14'-6"	10'-0"	2'-0"	6'-0"	5'-0"	1'-0"	36'-0"	18'-0"
I	3/8"	14'-6"	10'-0"	2'-0"	6'-0"	5'-0"	1'-0"	36'-0"	18'-0"
J	3/8"	14'-6"	10'-0"	2'-0"	6'-0"	5'-0"	1'-0"	36'-0"	18'-0"
K	3/8"	14'-6"	10'-0"	2'-0"	6'-0"	5'-0"	1'-0"	36'-0"	18'-0"
L	3/8"	14'-6"	10'-0"	2'-0"	6'-0"	5'-0"	1'-0"	36'-0"	18'-0"
M	3/8"	14'-6"	10'-0"	2'-0"	6'-0"	5'-0"	1'-0"	36'-0"	18'-0"
N	3/8"	14'-6"	10'-0"	2'-0"	6'-0"	5'-0"	1'-0"	36'-0"	18'-0"
O	3/8"	14'-6"	10'-0"	2'-0"	6'-0"	5'-0"	1'-0"	36'-0"	18'-0"
P	3/8"	14'-6"	10'-0"	2'-0"	6'-0"	5'-0"	1'-0"	36'-0"	18'-0"
Q	3/8"	14'-6"	10'-0"	2'-0"	6'-0"	5'-0"	1'-0"	36'-0"	18'-0"
R	3/8"	14'-6"	10'-0"	2'-0"	6'-0"	5'-0"	1'-0"	36'-0"	18'-0"
S	3/8"	14'-6"	10'-0"	2'-0"	6'-0"	5'-0"	1'-0"	36'-0"	18'-0"
T	3/8"	14'-6"	10'-0"	2'-0"	6'-0"	5'-0"	1'-0"	36'-0"	18'-0"
U	3/8"	14'-6"	10'-0"	2'-0"	6'-0"	5'-0"	1'-0"	36'-0"	18'-0"
V	3/8"	14'-6"	10'-0"	2'-0"	6'-0"	5'-0"	1'-0"	36'-0"	18'-0"
W	3/8"	14'-6"	10'-0"	2'-0"	6'-0"	5'-0"	1'-0"	36'-0"	18'-0"
X	3/8"	14'-6"	10'-0"	2'-0"	6'-0"	5'-0"	1'-0"	36'-0"	18'-0"
Y	3/8"	14'-6"	10'-0"	2'-0"	6'-0"	5'-0"	1'-0"	36'-0"	18'-0"
Z	3/8"	14'-6"	10'-0"	2'-0"	6'-0"	5'-0"	1'-0"	36'-0"	18'-0"

SHAPE OF BAR	

SHAPE OF BAR	

SHAPE OF BAR	

SHAPE OF BAR	

SHAPE OF BAR	

SHAPE OF BAR	

SHAPE OF BAR	

WALL SLAB									
NAME	SIZE	A	B	C	D	E	F	G	H
H	3/8"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"
I	3/8"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"
J	3/8"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"
K	3/8"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"
L	3/8"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"
M	3/8"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"
N	3/8"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"
O	3/8"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"
P	3/8"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"
Q	3/8"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"
R	3/8"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"
S	3/8"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"
T	3/8"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"
U	3/8"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"
V	3/8"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"
W	3/8"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"
X	3/8"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"
Y	3/8"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"
Z	3/8"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"	3'-4"	3'-25"

SHAPE OF BAR	

SHAPE OF BAR	

SHAPE OF BAR	

SHAPE OF BAR	

SHAPE OF BAR	

SHAPE OF BAR	

SHAPE OF BAR	

Load Test Conducted on Bridge AT 1/8 OF ODDANCHATRAM-DHARAPURAM ROAD

(Contributed from Highways Research Station)

SYNOPSIS

This article describes the load test conducted on bridge at 1/8 of Oddanchatram — Dharapuram Road in Madurai District Madras State. The reinforcement bars in the tee beams of the bridge had been joined by gas welding and so the test was conducted on all the spans with loads equivalent to the design load in order to fix the capacity of the structure.

The distribution of load in the various members of the bridge which is one of the important problems confronting the designs engineer in the examination of the load carried by main girders of a bridge deck, is discussed.

The procedure adopted in conducting the test, the method of loading, the position of loading and the effect of temperature on the beams are described with suitable illustrations.

The results of the test, the conclusions derived and certain suggestions of general nature for future guidance are also given.

INTRODUCTION

1. 1. The road from Oddanchatram to Dharapuram is an important Major District Road in Madurai District connecting two marketing centres, Oddanchatram and Dharapuram. The river Nanganjiyar crosses this road at mile 1/8 and a bridge with 3 vents of 40 feet was constructed at 1/8 of Oddanchatram—Dharapuram Road during the year 1950. The location of the bridge is indicated in the key map. Fig No. 1.

1. 2. The foundations of the bridge are taken down to 67'00 i.e. about 15 ft. below bed level. The abutment and piers are constructed in coursed rubble masonry in cement mortar.

1. 3. The superstructure consists of 4 tee beams, simply supported at ends and placed at 6'-0" centre to centre without interconnecting diaphragms. The size of the rib is 43" x 15" with a slab thickness of 7" and the road width between the kerbs is 22 feet. The main reinforcements of the Tee beam consists of 8 numbers of $1\frac{1}{2}$ " dia bars placed in two rows of 4 numbers each with a cover of $1\frac{1}{2}$ ". The details of the bridge and the reinforcements are given in Fig. No. 2.

PRESENT CONDITION UNDER WHICH THE TEST IS CONDUCTED

2. 1. Prior to the issue of the orders prohibiting gas welding it appear that the reinforcements bars in the tee beams of the bridge had been joined by gas welding. The details of the welding of the reinforcements rods are given in Fig. No. 3. The coupling adopted also does not conform to the standard (design furnished below) and the sample of welded rods have not been tested. It is usual to place the joints staggered and away from the section where Maximum Bending Moment occurred so as to avoid a particularly weak section but in this case the location of the joints are also not known. Under these circumstance it became necessary to ascertain the capacity of the structure for sustaining design loads. As deflection is a measure of the elasticity of the structural member and gives an idea of the capacity of the structure for sustaining loads when compared with calculated deflections, a load test was conducted on all the spans with loads equivalent to the design load so as to fix the capacity of the structure.

2. 2. This opportunity was also utilised for finding the distribution of the load in the various members of the bridge which is one of the important problem confronting the designs Engineer in the examination of the load carried by main girders of a bridge deck. The usual specification for estimating the load carried by main girder is based on empherical distribution. For example A.A.S.H.O. specification require an intermediate beam to be designed for wheel load equivalent to spacing of the girders divided by 5. For exterior girders the line load is assumed to be the reaction of the wheel load. It has become a common practice among the designers to make the end beams less strong than the interior ones. The I.R.C. specification does not throw much light on the distribution of load and is more in line with the above mentioned specifications.

2. 3. It is a common knowledge that when the main beams are loaded a portion of the load is transmitted to the adjoining beams but what exact percentage of the total load is shared by intermediate and end beams of the structure has long been a problem. Many investigations have tried to solve this basing on various assumptions as the variables involved are many, analytical solution of the problem is difficult.

2. 4. There will not be much variations of E and I for the various beams of the bridge and as such the deflection under each beam will be proportional to the load taken by the particular beam. Hence with a view of investigating the load distribution in the different beams a systematic test was also conducted for ascertaining the load taken by each beam under different conditions of loadings.

PROCEDURE-INSTRUMENTATION

3. 1. The effect of the load test on the members of the structure is observed by the examination of the number during and after test by measurement of the deflections produced by the loading. Sensitive dial gauges were used for recording the deflections of the beams. Timber trestle platforms were erected below the centre of the T beams in all the span for fixing up the dial gauges. These platform consists of two rows of stout straight wooden posts fixed vertically and held in position with suitable batters as illustrated in the sketch shown in Fig No. 4. A wooden uniform sized cross beam is fixed at right angles to the Tee beams. The dial gauges were tightly fixed to the instrument platform by means of wooden clamps (Photo 3). Observers platform were also erected at convenient heights for observing the dial gauge readings. Instrument platforms and observers platform were also erected at the quarter points in the first span of the bridge to observe the deflections during the distribution tests. The instrument platform and observers platform and the dial gauges are illustrated in photos No. 1 and No. 2. Both Batty and Mercer type dial gauges with an accuracy of 1/2000 inch were made use of during the load test. Iron clamps were closely fitted for the Tee beams at the points where it was proposed to measure the deflections in order to provide an even surface for the spindle of dial gauges.

METHOD OF LOADING

3. 2. It is difficult to make the test load to be exactly same as the specified design load which may be wheel or track loads. The bridge tested is designed for

I. R. C. class A loading and the maximum live load conditions is converted for the equivalent uniformly distributed load. Please *vide* Appendix I for the details. Sand bags of uniform size and of uniform weight were arranged in independent vertical piles with clearance between each pile so as to prevent any possible 'arching' action with in the loading material thus preventing unequal distribution of load. The weight of each sand bag was so adjusted as to have eight numbers in each pile in the loading area which facilitated for applying 1/4, 1/2, 3/4 and full design load gradually and systematically so that the loading may be restricted in case the member tested showed any signs of distress.

POSITION OF LOADING

3. 3. For testing intermediate beams equivalent uniformly distributed load was applied from the centre of the first beam to the centre of the third beam. Consecutive loadings were done in alternate spans so that the effect of recovery was also noted when the loads were removed. For testing end beams load was applied on the cantilever portion of slab and half the end bay. The loading area and the sequence of loading are shown in Fig. No. 5.

POSITION OF LOADING FOR DISTRIBUTION TESTS

3. 4. For finding the load distribution in the various members of the structure a systematic loading was done with single roller, two rollers, single lorry, two lorries and the combination of the roller and lorry in various positions as noted below.

I. Symmetric Loadings :

- (a) Single roller symmetrical to bridge axis
- (b) Two rollers symmetrical to bridge axis
- (c) Single lorry symmetrical to bridge axis
- (d) Two lorries symmetrical to bridge axis
- (e) Combination of a lorry and roller with centre of gravity in the centre of bridge axis

II. Eccentric Loadings :

- (1) Single roller eccentric to bridge axis
- (2) Two rollers eccentric to bridge axis
- (3) Single lorry eccentric to bridge axis
- (4) Two lorry eccentric to bridge axis
- (5) Combination of a roller and lorry eccentric to bridge axis

The position of loadings are illustrated in the plates No. 1 to 8.

OBSERVATION AND READINGS

4. 1. Before commencing the loading operations the instruments were fixed and ensured that the deflectometers are in contact with the structure and the initial readings are carefully noted and entered. The temperature and time were also observed and noted.

During a thorough and careful examination of the numbers before the commencement of the loading a minute longitudinal crack was observed in the centre of the middle span. Thin tissue paper was pasted right through the crack to find out any further openings of

it due to the load during the test. The paper was found in tact even when full load was applied and perhaps it may be a constructional crack in the slab and need not be considered as detrimental.

TEMPERATURE

4. 2. The effect of the temperature variations is prominent in a composite structure like the reinforced concrete beam and hence it will considerably affect the measurement of deflections due to loads. Observations for deflections caused by temperature were made every 30 minutes to find the behaviour of the beams under temperature variations without any load on the structure. The temperature variations between the upper and lower points of the cross section of the members tested were also recorded by keeping two thermometers one in the shade and the other exposed to the sun. From the temperature variations and the resulting deflections under no load, suitable corrections were effected to the dial gauge readings noted under test load since the measured readings are not only due to test load but also due to the temperature effect on the various members. The effect of temperature on the beams is illustrated in the Graph No. 1.

The load was applied steadily and systematically and the deflectometer readings were recorded at intervals of every quarter load. The structural members were carefully observed at critical sections for the development of any possible cracks or other sign of weakness. The full load was allowed to remain in the position for a considerable time before recording the dial gauge readings for maximum deflection. Residual

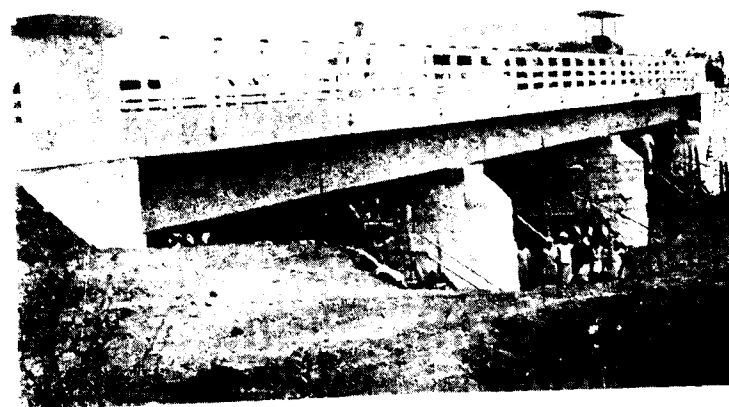
or permanent deflections after the removal of half load and full load were also noted for some of the members tested. (Graph No. 2.)

MAXIMUM DEFLECTION (THEORETICAL)

4. 3. The deflection of reinforced concrete members cannot be calculated with precision owing to such variable factors as E, shrinkage in concrete and creep effects etc. but this is not of serious consequence since the indefinite numerical values offset each other to a large extent. The amount of the actual maximum deflection was calculated for a uniform distributed load from $(5/384) \frac{WL^3}{EI}$ where W is the total load on the beam L is the length of the span and E modulus of elasticity of concrete and I the equivalent moment of Inertia. Reynolds has suggested an approximate value of 37,50,000 for E corresponding to a modular ratio of 8. It would be appropriate to give a higher value for E since it is more than 5 years since the construction was completed and concrete would have set considerably and a value of 4×10^6 was adopted in the deflection calculations. The moment of Inertia I was calculated for the combined section. *Vide* the appendix for the detailed calculations.

INTERPRETATION OF RESULTS

5. 1. Deflection corresponding to the designed load is a measure of the elasticity of the structural member and hence gives an idea of the capacity of the structure for sustaining loads. The theoretical deflections for the designed load is calculated in the appendix. The maximum deflection under full load for the various beams was found comparatively less than the calculated deflection as given in the Appendix.



A view of the bridge under test

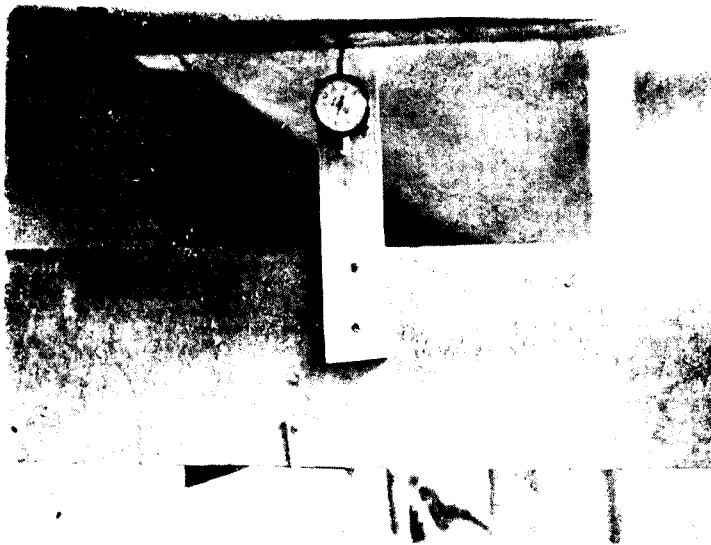


The increase in the deflection was observed to be in proportion to the rate of loading and similarly the rate of recovery was also proportional to the rate of release of the load. The recovery in some of the cases was found to be more than 90% of the maximum deflection. The record of observation is given separately.

DISTRIBUTION TESTS

5. 2. The test results of the distribution tests under systematic loading revealed that the end beams also shared certain amount of the total load even though no load was applied over that. The load distribution for a 10 ton roller in a symmetric position was 14% in end beams of and 36% in the intermediate beams as shown in Plate 3. It was observed that about the same distribution of load occurred even when a single lorry or a combination of 2 rollers or 2 lorries were placed symmetrical to bridge axis. Under eccentric loading the end beams took more load then the intermediate beams. The end beam shared nearly 57% of the load when a single roller was placed eccentric as shown in plate. When a roller and a lorry were placed such that the centre of gravity of system coincided with the axis of the bridge the distribution of load was noticed to be similar to the cases of two rollers or two lorries in symmetrical position mentioned above.

When the C. G. of the system was shifted towards the curb the end beam deflected more revealing that it shared more load as the point of application of loads was shifted towards that beam. The greater the eccentricity of the load greater the percentage of load shared by end beam. The roller combination series with varying position of the eccentricity showed the



Dial Gauge

percentage of the load taken by end beam can be reduced to the form of an equation $p = mx^n + c$ as shown in (Graph No. 3) [$P = 2.09 x^{1.47} + 16.6$]. Where p is the percentage of load and m , n and c are constants.

CONCLUSIONS

Full Live Load Test:

6. 1. (a) The full live load test revealed that the observed deflections were much less than the calculated deflections which showed the tee beams are stiff for carrying the design loads.

(b) The recovery of deflection was nearly 90 percent. The maximum recovery stipulated as per building code of American Concrete Institute should be not less than 75%.

(c) The test load was perhaps the maximum load that the bridge would be subjected as the test load was equivalent of full live load stipulated as per I.R.C. class A two lanes with impact. The structure did not show any signs of distress while loading or unloading.

(d) In view of the above considerations the structure can be considered to be quite safe for carrying the I.R.C. A class loading for two lanes.

LOAD DISTRIBUTION TESTS

6. 2. (i) It is seen that all the beams deflected whatever be the position of load and this shows the slab is stiff and distributes loads.

(ii) When the loads are placed symmetrical to bridge axis the intermediate beams share more load than end beams.

(iii) When the loading system is eccentric to bridge axis there is a tendency for end beam to share more and more load as the point of application of load is moved towards it from bridge axis.

(iv) The load that is shared by the end beam for the conditions of this structure appears to obey the law $P = 2.09 x^{1.47} + 16.6$ where x is eccentricity and P the percentage of load taken by end beam.

(v) It is noticed that the end beams share loads as much as intermediate beams and even more than intermediate beams in certain conditions of loading. Hence there appears to be no case to make end beams less strong than intermediate beams and it may not be correct to curtail reinforcements in end beams on the assumption that end beams are taking load only of that wheel load which is directly on the beam or in the last bay.

GENERAL

7. 1. While the above conclusions on the load shared by individual beams, no doubt is indicative of the general applicability yet it must be stated that the conclusions cannot be applied in toto, each case requiring modifications due to such varied factors such as whether the girders are simply supported whether the bridge is right or skew, the number of girders, spacing of girders, the number location and type of diaphragms etc., etc. A few more tests on tee beam

bridges of varied types incorporating these features may be necessary to come to a more decisive conclusion.

APPENDIX—I

Calculation for the equivalent distributed load

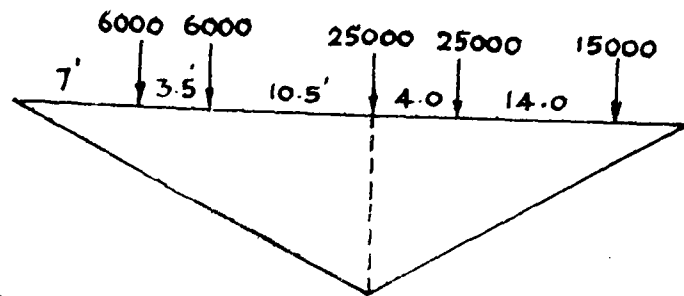
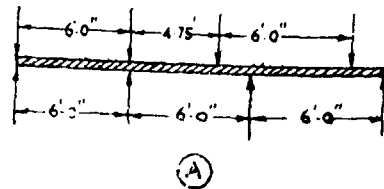
Span = 42 feet

$$\text{Impact factor} = \frac{15}{20 + L} = \frac{15}{20 + 42} = \frac{15}{62} = 0.242$$

Wheel load reaction on the interior beam for I. R. C. class A loading Reaction on A due to 2 lane traffic

$$W + \frac{1.25}{6.00} W = 1.21 W$$

$$\text{With impact factor reaction} \\ 1.21 W \times 1.242 = 1.502 W$$



Influence line for midspan

$$\text{Ordinate} = 1/2 \times 1/2 \times 42 = 10.5$$

$$\text{B.M.} = 1.502 \left[3000 (7 + 10.5) \frac{10.5}{21} + 1250 (21 + 17) \frac{10.5}{21} + 7500 \times 3 \times \frac{10.5}{21} \right]$$

$$= 1.502 \times \frac{10.5}{21} (3000 \times 17.5 + 12500 \times 38 + 7500 \times 3 \times \frac{10.5}{21})$$

$$= 1.502 \times \frac{10.5}{21} (52500 + 4,75,000 + 22,500)$$

$$= 1.502 \times \frac{10.5}{21} \times 5,50,000 = 413200 \text{ ft. lb. or } 184.5 \text{ ft. tons}$$

Maximum live load Bending moment at the end = 184.5 ft. tons

Maximum bending moment for uniformly distributed load

$$= \frac{W L^2}{8} = 184.5 \text{ ft. ton}$$

W = uniformly equivalent distributed load = 78700 lbs.

APPENDIX—II

Calculation for the number of sand bags required for loading

Loading area for a single beam = 40' - 0" × 6' - 0"

The size of the sand bags is nearly 1' - 9" × 1' - 3"

5 rows of 21 bags can be placed conveniently with enough space in between them

Number of bags per layer = 5 × 21 = 105

It is desired to have eight layers for the full load

Total number of bags for eight layers 105 × 8 = 840 bags

$$\text{Weight per bag} = \frac{78700}{840} = 94 \text{ lbs.}$$

APPENDIX—III

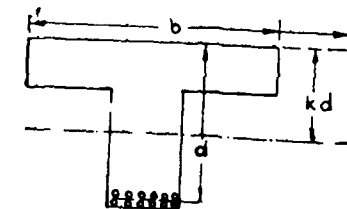
2. Calculation of moment of inertia of the concrete steel section the steel area being weighted n fold

$$I = bd^3 \left[k^3 - \left(1 - \frac{b'}{b}\right) \left(k - \frac{t}{d}\right)^2 + \frac{b'}{b} \left(1 - k^3\right) + 3 p n (1 - k)^2 \right]^{1/3}$$

$$d = 45.75"$$

$$kd = 16$$

$$k = \frac{16}{45.75} = 0.35"$$



$$\frac{b'}{b} = \frac{15}{72} \quad p = \frac{A_s}{bd} = \frac{14.14}{72 \times 45.75} \times 0.004$$

$$I = 72 \times 45.75^3 \left[0.35^3 - \left(1 - \frac{15}{72}\right) \left(0.35 - \frac{7}{45.75}\right)^2 + \frac{15}{72} (1 - 0.35)^3 + 3 \times 0.004 \times 15 (1 - 0.35)^2 \right]^{1/3}$$

$$= \frac{72}{3} \times 45.75^3 \left[0.35^3 - (8) (0.197)^2 + .2 (0.65)^3 + .025 \right]$$

$$= 24 \times 45.75^3 [0.043 + .006 + .05 + .025] \\ = 2410000 \times 0.124 = 2,98,840 \text{ in}^4$$

APPENDIX—IV

Calculation of the maximum deflection

$$\frac{5}{384} \times \frac{WL^3}{EI}$$

Uniformly distributed load = 78700 = W and L = 42 ft. = 504 inch and $E_c = 4 \times 10^6$

$$= \delta = \frac{5}{384} \times \frac{78700 \times 504^3}{40 \times 10^6 \times 2,98,840} = .0878''$$

APPENDIX—V

Table showing the maximum observed deflection and the permissible deflection

Serial No.	Span No.	Position of loading	Position of beam	Max. observed deflection	Maximum Permissible deflection	Remarks
1	First span ...	No. 1	Central beam	0.076"	0.0878"	
2	Second span...	No. 2	Do.	0.073"	Do.	
3	First span ...	No. 3	Do.	0.069"	Do.	
4	Centre span ...	No. 4	Do.	0.066"	Do.	
5	First span ...	No. 5	End beam	0.037"	Do.	
6	Second span ...	No. 6	Do.	0.045"	Do.	
7	First span ...	No. 7	Do.	0.053"	Do.	
8	Centre span ...	No. 8	Do.	0.044"	Do.	
9	Third span ...	No. 9	Centre beam	0.072"	Do.	
10	Do. ...	No. 10	Do.	0.082"	Do.	
11	Do. ...	No. 11	End beam	0.046"	Do.	
12	Do. ...	No. 12	Do.	0.035"	Do.	

RECORD OF OBSERVATION

Uniformly distributed load—First span—Centre point—Loading position No. 3

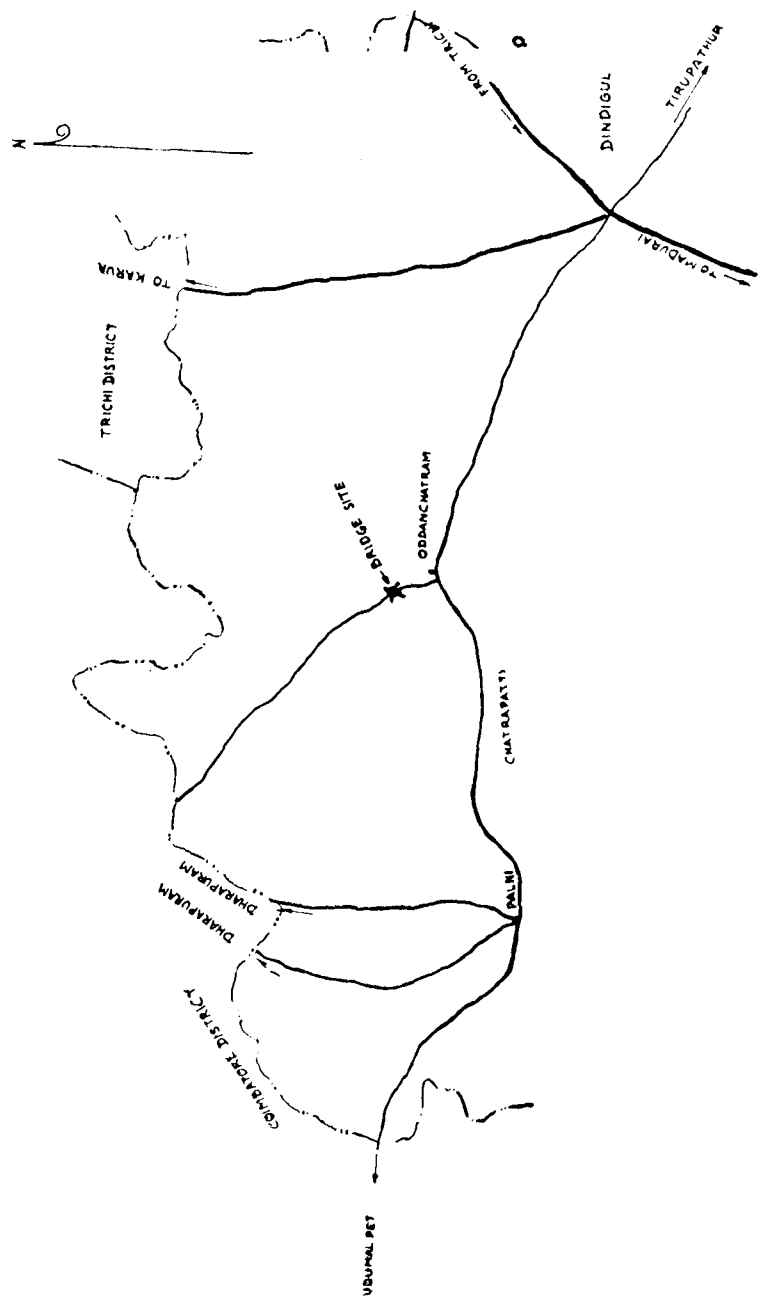
Load condition and dial gauge positions	Time	Temp. in 0° F.	Initial reading		Final reading		Difference		Deflection in 1/1000 inch	Remarks
			Main Dn.	Sub Dn.	Main Dn.	Sub Dn.	Main Dn.	Sub Dn.		
25--8--1956										
1/4 Load B 1 B 2 B 3 B 4	9.20 A.M.	82	0	0	0	10	0	10	10	
	"	"	1	34.5	1	47	0	12.5	12.5	
	"	"	1	0	1	9	0	9	9	
	"	"	0	0	0	2.5	0	2.5	2.5	
1/2 Load B 1 B 2 B 3 B 4	9.45 A.M.	83	0	10	0	20	0	10	20	
	"	"	1	47	2	9.5	0	12.5	25	
	"	"	1	9	1	19	0	10	19	
	"	"	0	2.5	0	5.5	0	3	5.5	
3/4 Load B 1 B 2 B 3 B 4	10.15 A.M.	84	0	20	0	30	0	10	30	
	"	"	2	9.5	2	22.5	0	13	38	
	"	"	1	19	1	26	0	7	26	
	"	"	0	5.5	0	8	0	2.5	8	
1 Load B 1 B 2 B 3 B 4	11 A.M.	86	0	30	0	40	0	10	40	
	"	"	2	22.5	2	36	0	13.5	51.5	
	"	"	1	26	1	35	0	9	35	
	"	"	0	8	0	11	0	3	11	

RECORD OF OBSERVATION

Distribution Test—First Span—Centre Point

Load Condition	Time	Temp in 0° F.	Initial reading		Final reading		Difference		Deflec- tion in 1/1000 inch	Remarks
			Main Dn.	Sub Dn.	Main Dn.	Sub Dn.	Main Dn.	Sub Dn.		
26-8-1956										
Single Lorry (10 tons)	12 Noon	89	0	0	0	4.5	0	4.5	4.5	
Symmetric	"	"	0	0	0	11	0	11	11	
B 1 ...	"	"	0	0	0	11.5	0	11.5	11.5	
B 2 ...	"	"	0	0	0	4.5	0	4.5	4.5	
B 3 ...	"	"	0	0	0	14.5	0	14.5	14.5	
B 4 ...	"	"	0	0	0	12	0	12	12	
Single Lorry Eccentric	12.5 P.M.	89	0	0	0	1	0	1	1	
Right Side	"	"	0	0	0	1	0	1	1	
B 1 ...	"	"	0	0	0	2	0	2	2	
B 2 ...	"	"	0	0	0	4.5	0	4.5	4.5	
B 3 ...	"	"	0	0	0	6	0	6	6	
B 4 ...	"	"	0	0	0	15	0	15	15	
Single Lorry Eccentric	12.15 P.M.	89	0	0	0	13.5	0	13.5	13.5	
Left Side	"	"	0	0	0	19.5	0	19.5	19.5	
B 1 ...	"	"	0	0	0	19.5	0	19.5	19.5	
B 2 ...	"	"	0	0	0	13.5	0	13.5	13.5	
B 3 ...	"	"	0	0	0	13.5	0	13.5	13.5	
B 4 ...	"	"	0	0	0	13.5	0	13.5	13.5	
Two Lorries Symmetric	12-20 P.M.	90	0	0	0	8.5	0	8.5	8.5	
B 1 ...	"	"	0	0	0	16.5	0	16.5	16.5	
B 2 ...	"	"	0	0	0	21	0	21	21	
B 3 ...	"	"	0	0	0	18	0	18	18	
B 4 ...	"	"	0	0	0	18	0	18	18	
Two Lorries Eccentric	12-40 P.M.	90	0	0	0	18	0	18	18	
Right Side	"	"	0	0	0	21.5	0	21.5	21.5	
B 1 ...	"	"	0	0	0	17	0	17	17	
B 2 ...	"	"	0	0	0	8	0	8	8	
B 3 ...	"	"	0	0	0	10.5	0	10.5	10.5	
B 4 ...	"	"	0	0	0	19	0	19	19	
Combination of Roller and Lorry Centre of Gravity in Middle Axis	3-5 P.M.	88	0	0	0	10.5	0	10.5	10.5	
B 1 ...	"	"	0	0	0	19	0	19	19	
B 2 ...	"	"	0	0	0	17	0	17	17	
B 3 ...	"	"	0	0	0	8	0	8	8	
B 4 ...	"	"	0	0	0	10.5	0	10.5	10.5	
Combination of Roller and Lorry Maximum Eccentricity	3-15 P.M.	85	0	0	0	18	0	18	18	
B 1 ...	"	"	0	0	0	20.5	0	20.5	20.5	
B 2 ...	"	"	0	0	0	14	0	14	14	
B 3 ...	"	"	0	0	0	6	0	6	6	
B 4 ...	"	"	0	0	0	6	0	6	6	

KEY MAP SHOWING THE BRIDGE SITE—Scale: 1"=4 Miles



DETAILS OF THE BRIDGE ACROSS NANGANJIYAR—Scale: 1"=6 Feet

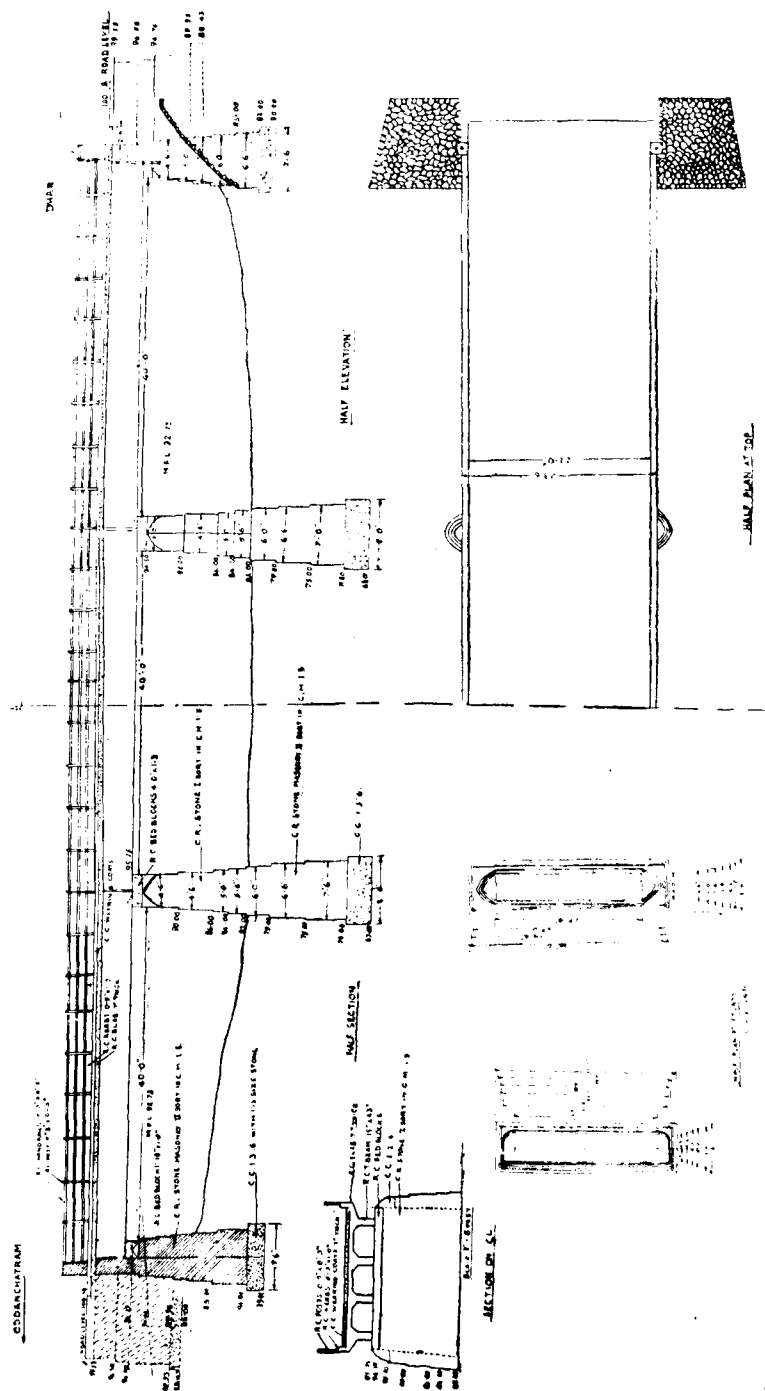
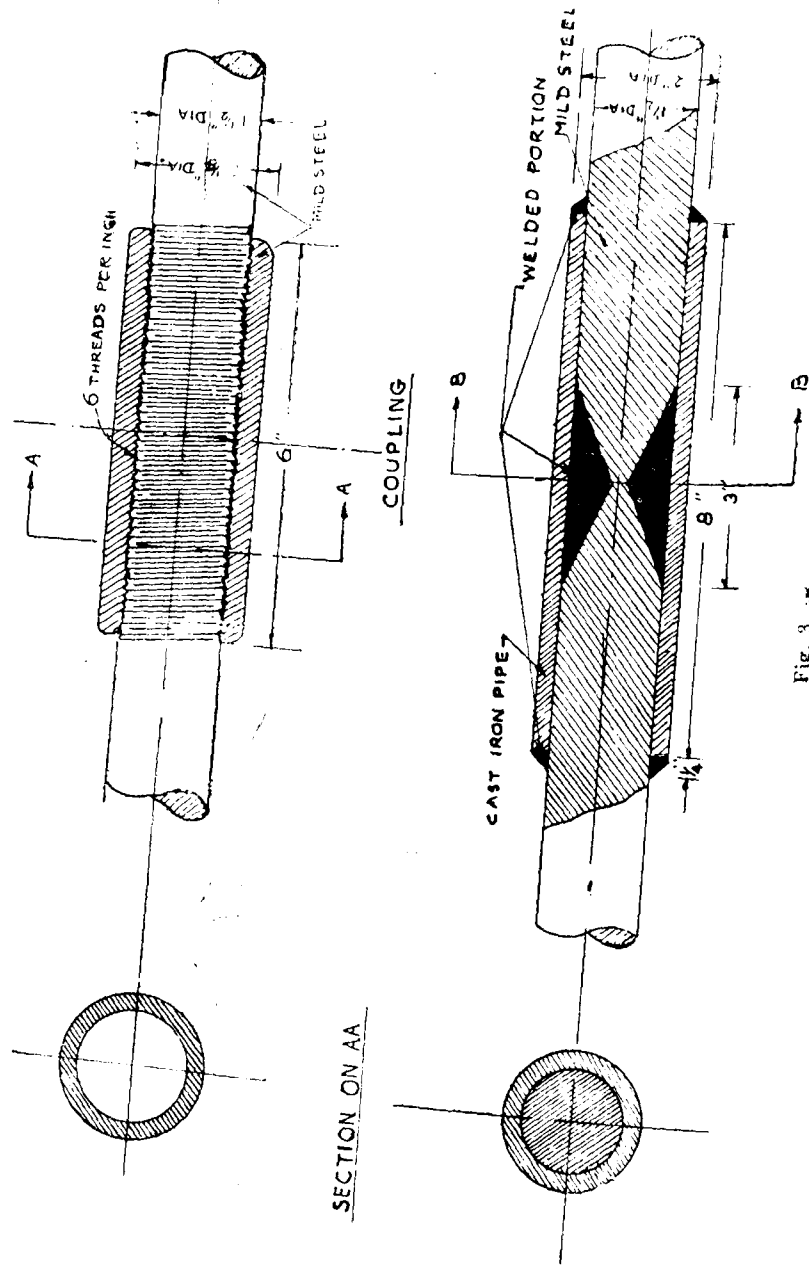
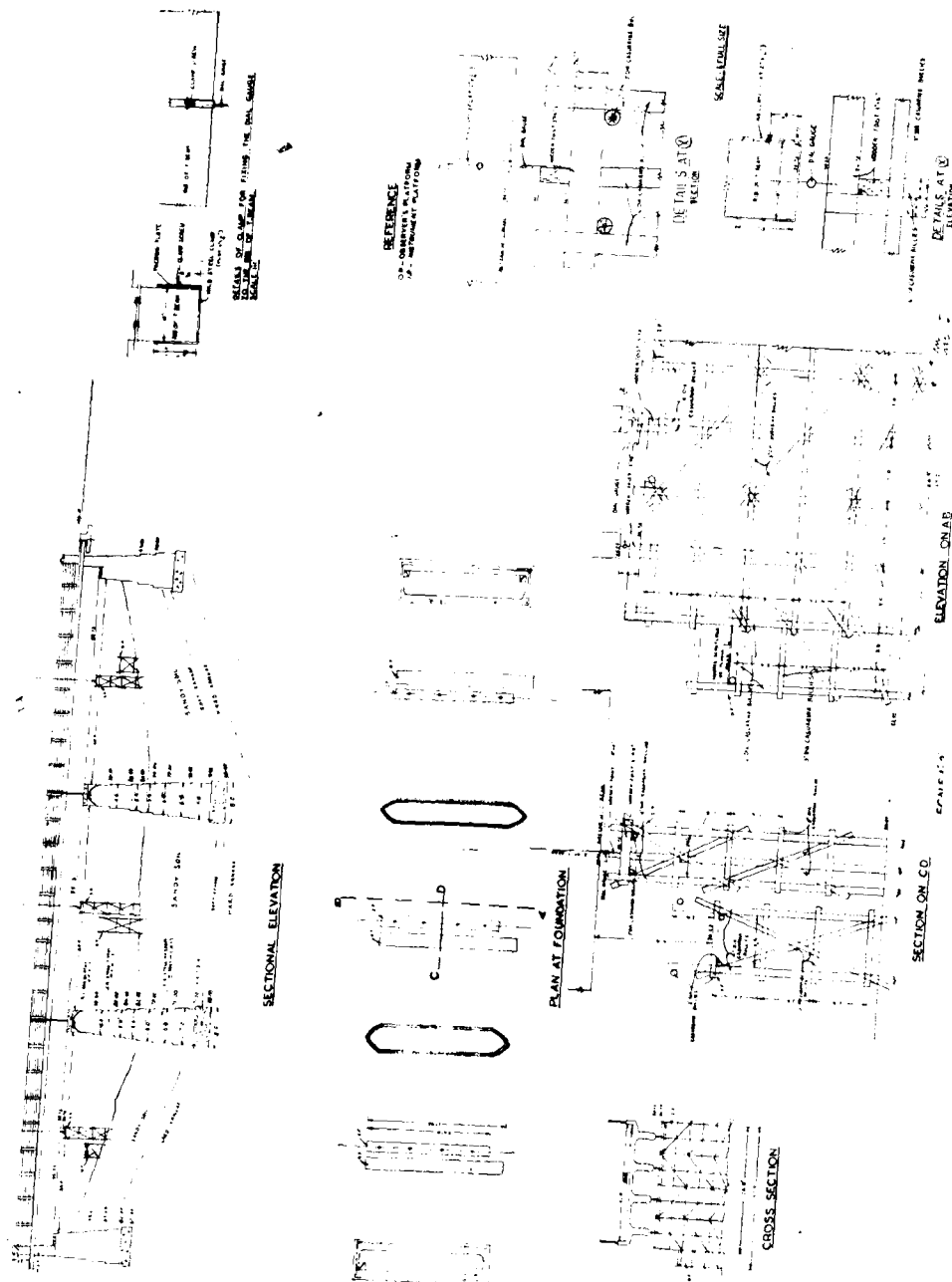


Fig. 2.

DETAILS OF JOINTS ADOPTED FOR 1 1/8" DIA. MAIN REINFORCEMENT
ROUNDS IN BRIDGE—Scale: Half Full Size



INDUCTING LOAD TEST ON THE BRIDGE AT MILE 8 OF ODDANCHATRAM DHARAPURAM ROAD.
TAILS OF OBSERVERS AND INSTRUMENT PLATFORM



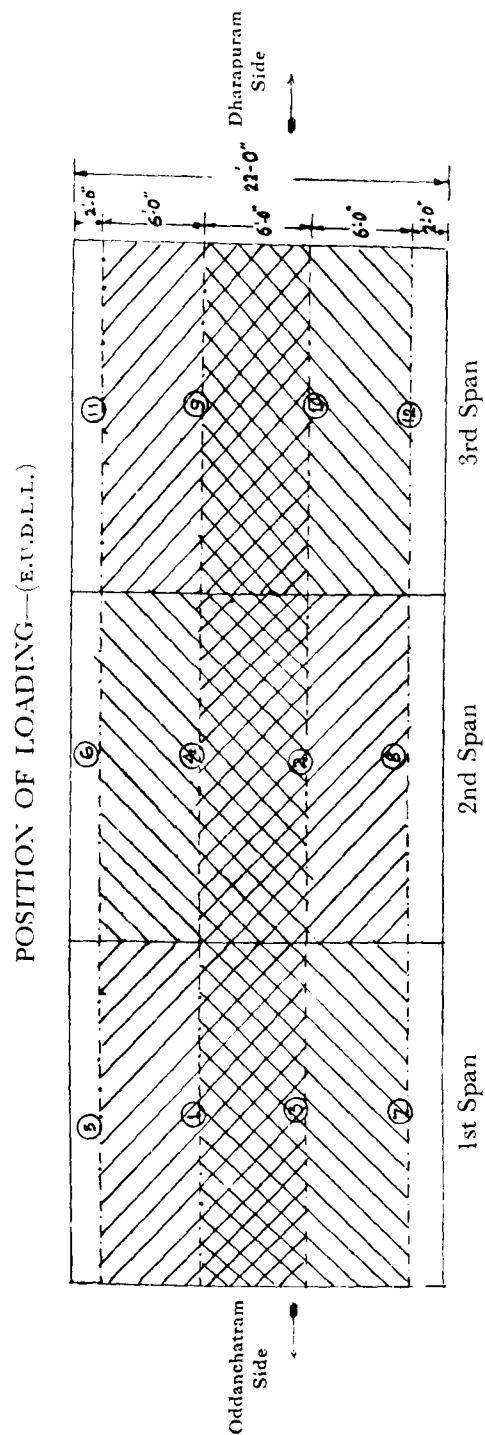


Fig. 5.

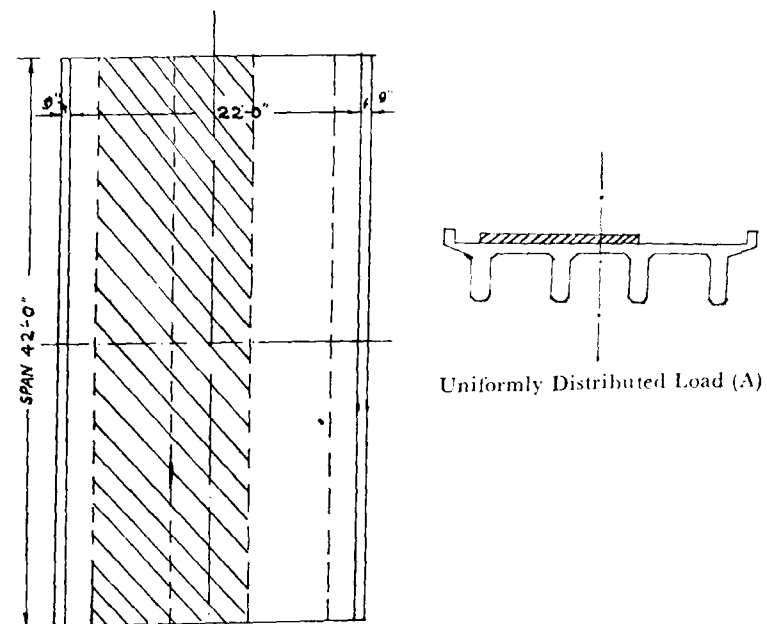


Plate No. 1

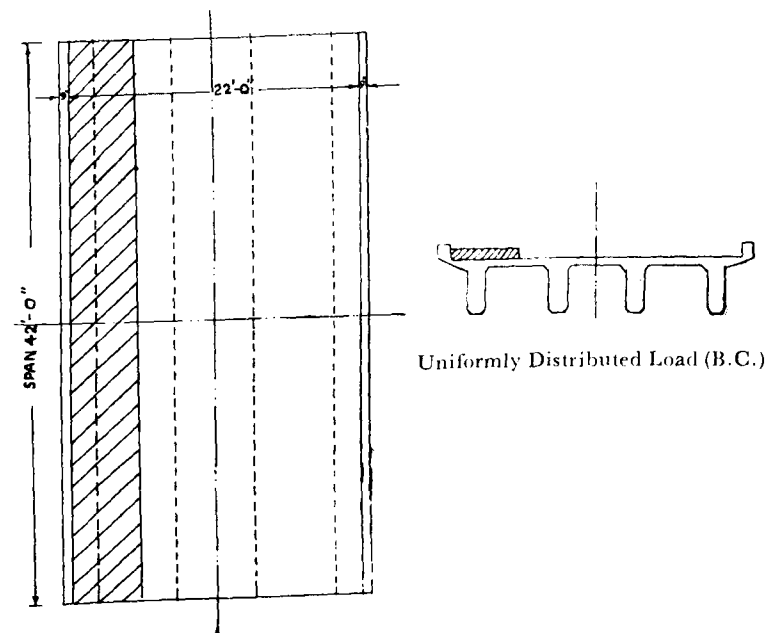


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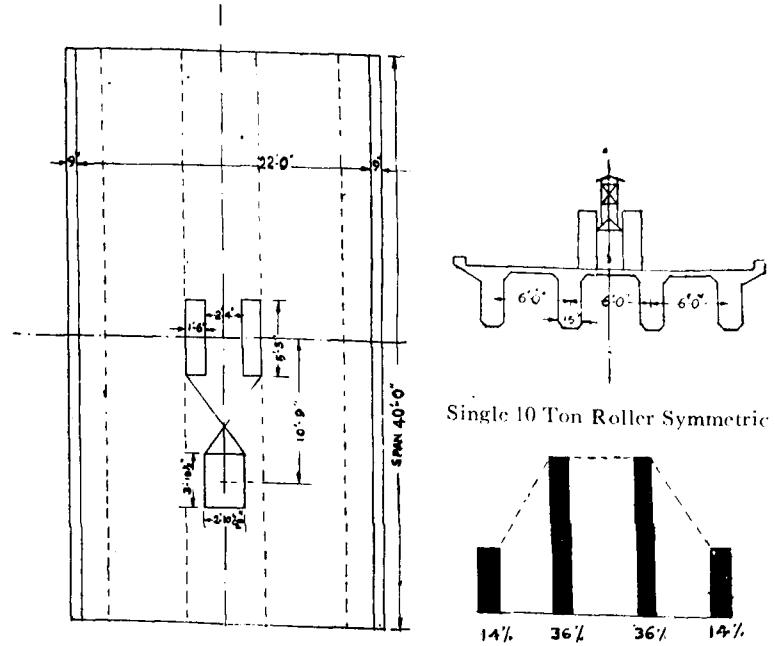


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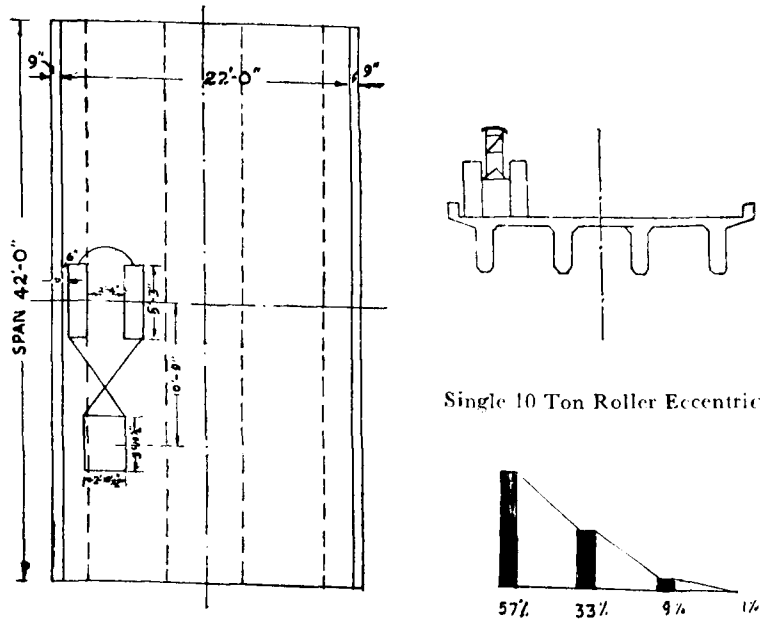


Plate No. 4

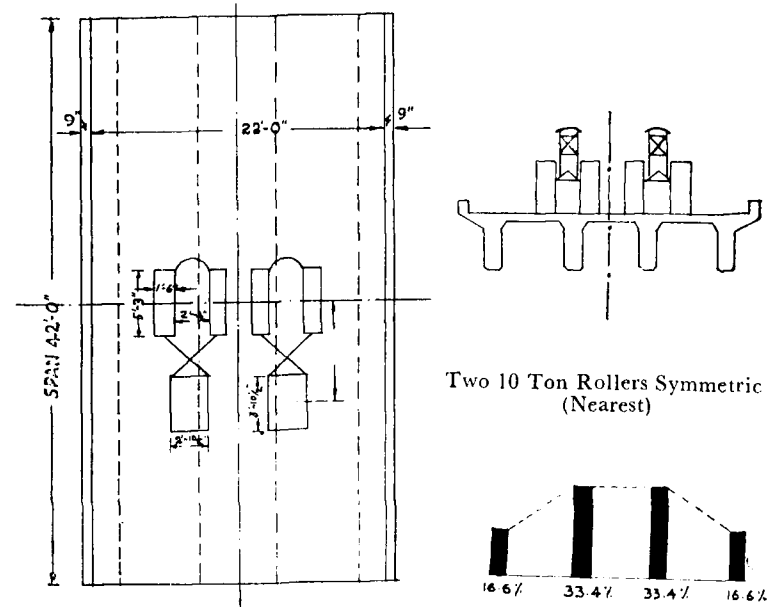


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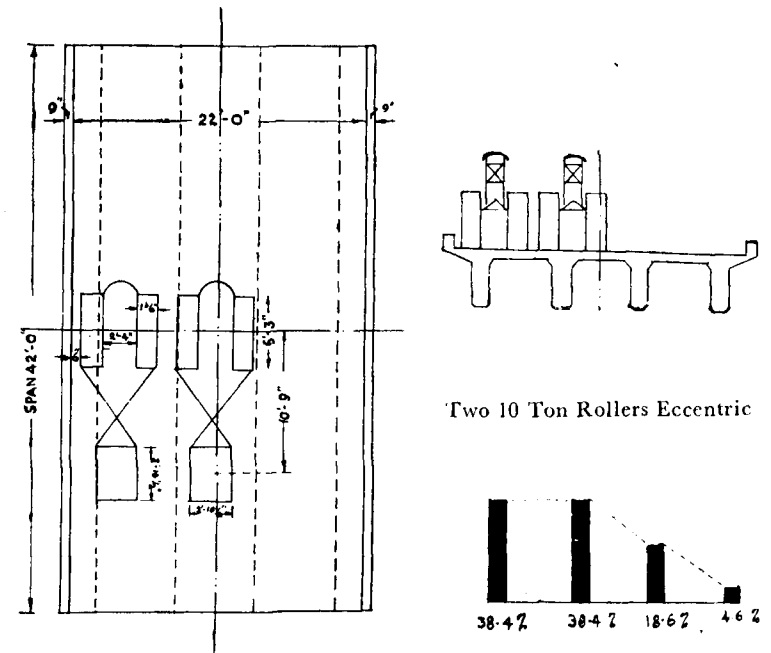
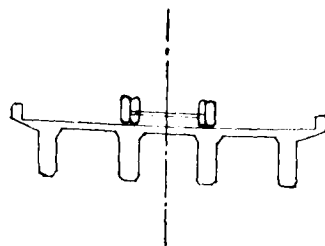
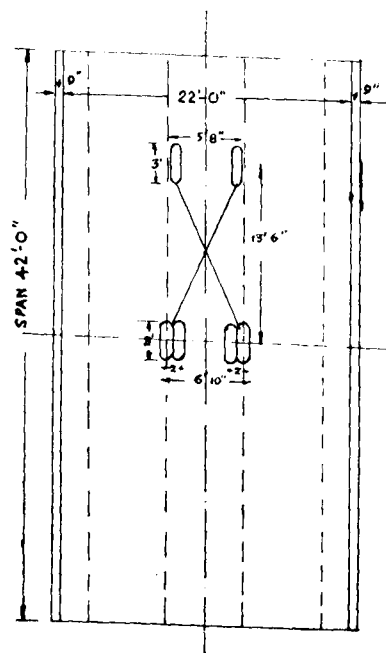


Plate No. 6



Single Lorry (3 Tons)
Symmetric

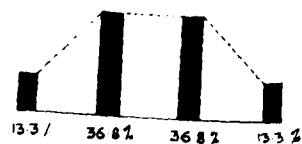
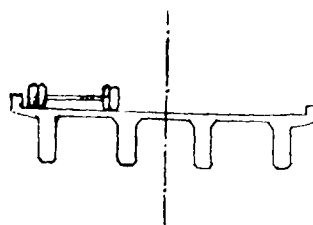
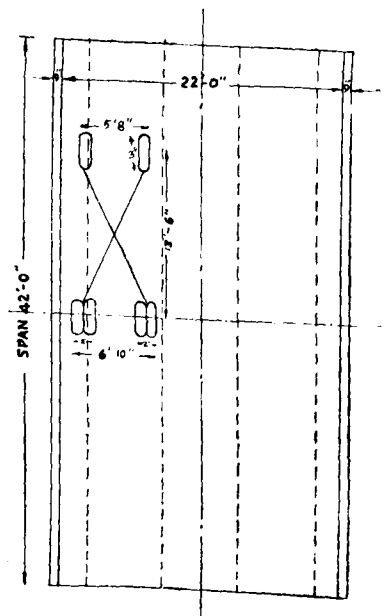


Plate No. 7



Single Lorry (3 Tons)
Eccentric

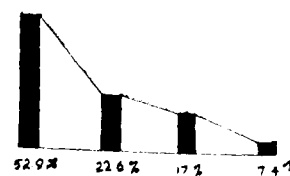
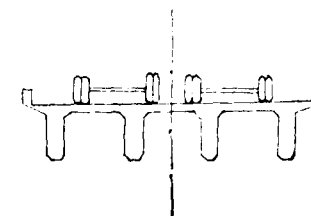
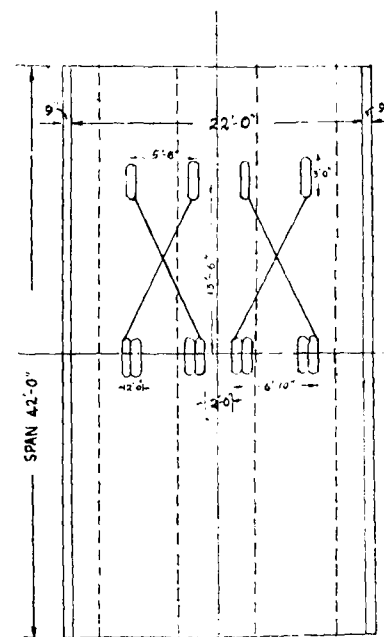


Plate No. 8



Two Lorries (3 Tons)
Symmetric

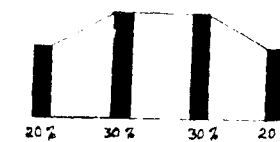
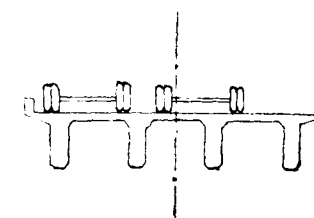
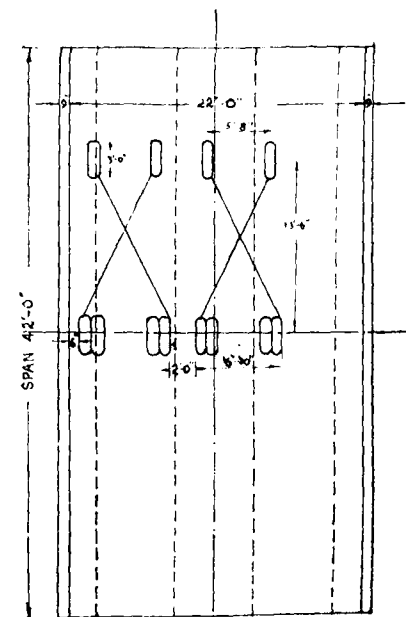


Plate No. 9



Two Lorries (3 Tons)
Eccentric



Plate No. 10

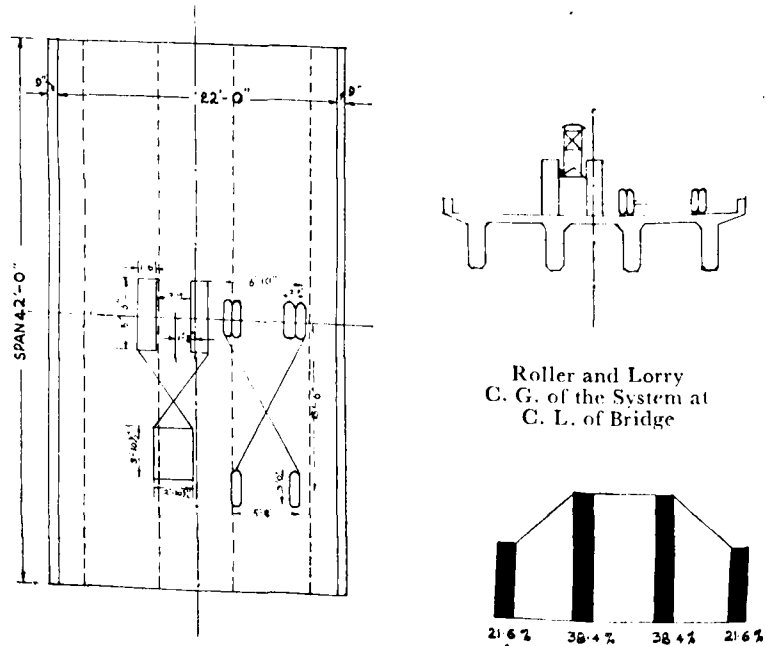


Plate No. 11

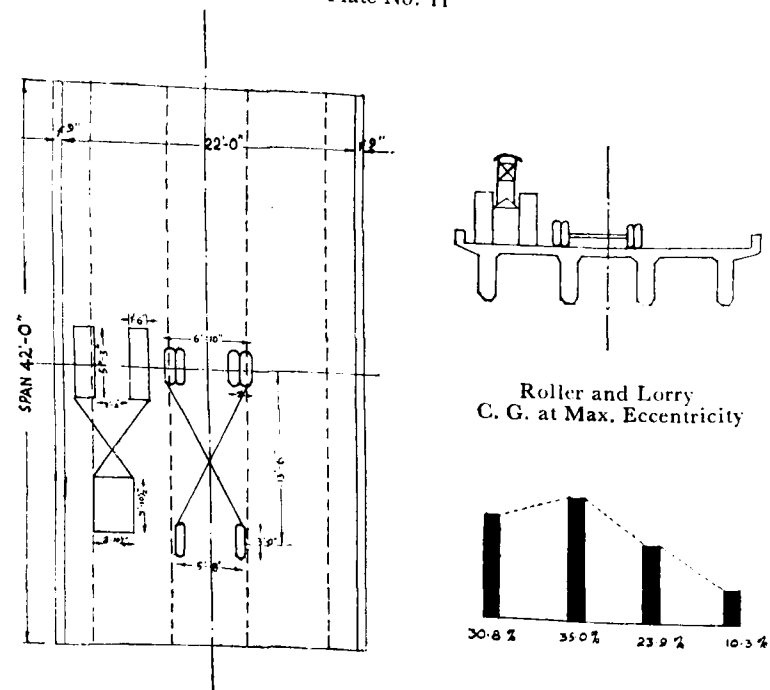
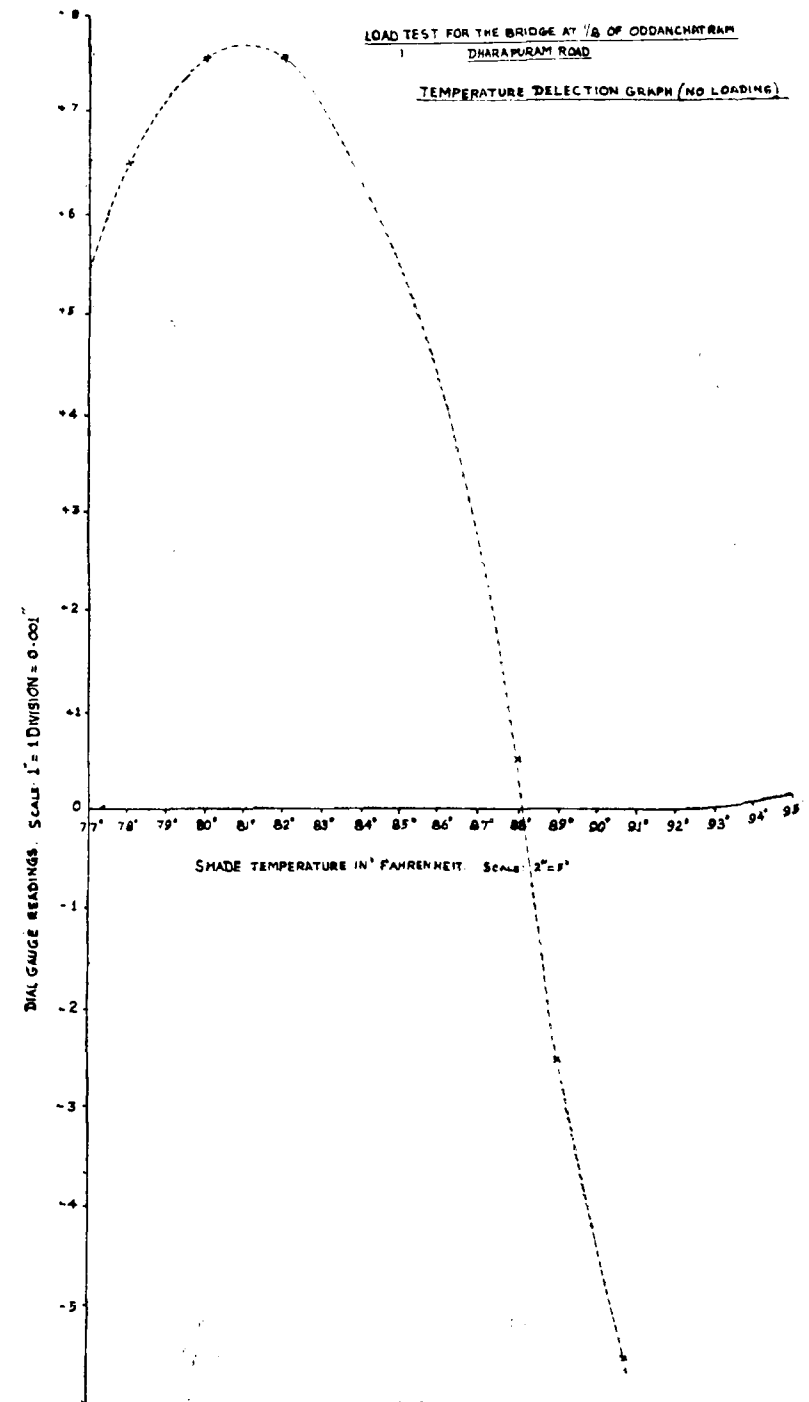
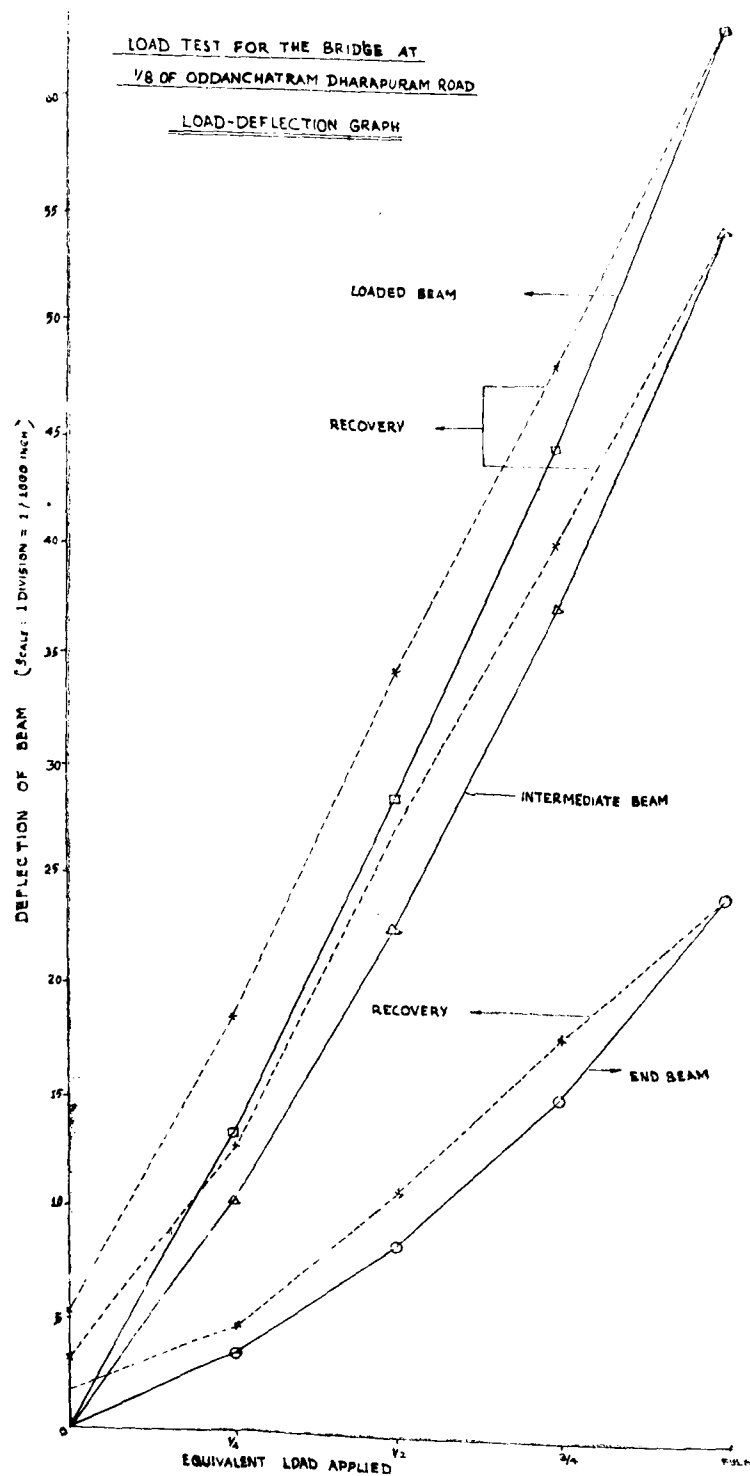


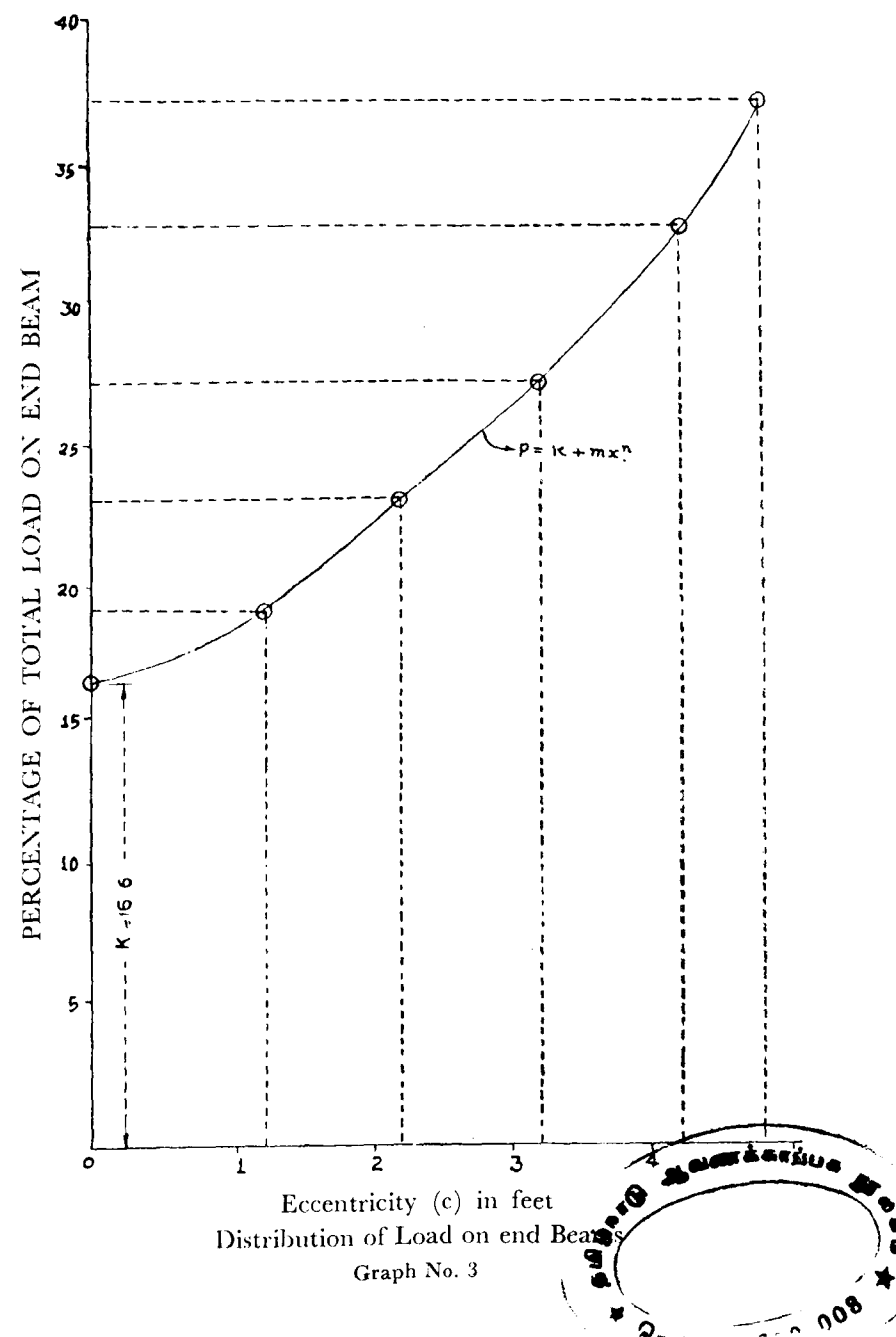
Plate No. 12



Graph No. 1



LOAD TEST FOR THE BRIDGE



R. C. Foot Bridge

BY

Sri V. SRINIVASAN, B.E.,

Junior Engineer, Highways, Kuttalam.

INTRODUCTION

The most common difficulty met with in all village communications which are frequently intercepted by small streams and channels, is the construction of a Bridge wherever the road crosses a stream channel etc. with minimum cost. Most of our village Roads may not experience, rather will not experience such a great volume of traffic as a pucca Main Road does. But many of them serves the purpose of transporting the grains etc. from village farms to cities. In all these cases, though a Bridge of the ordinary kind may be constructed, it becomes more costlier especially when the road crosses a number of streams which is the case with every Village Road lying in Deltaic area. For these, a wide bridge with Masonry piers is not necessary, but in the interest of economy, a bridge with sufficient roadway to allow the cart laden with grains to pass, constructed, on "Suitable Supports" is sufficient. Reduction in Roadway can be done only upto a certain limit which may not favour much to the economic aspect of the problem. But it is only the alternative arrangement of the supports over which the Bridge Rests, that effects maximum economy. The present article explains how this economy was effected in the R. C. Foot Bridge constructed across Thenkoraiyar at Koneri Rajapuram Village, Tanjore District of Madras State.

DEVELOPMENT OF TRANSPORT



CONSTRUCTION DETAILS

The R. C. Foot Bridge is of the ordinary Slab Type, designed to sustain a maximum Live Load of 3 Tons. The design features of the Slab are the same as usual, it being designed as a simply supported slab. The Roadway is 8'-0" clear between kerbs and the Bridge is of Two spans each of 10'-0" clear. All the other features of the superstructure such as Hand rails etc., are as usual. The only important deviation to be noted is, that the Bridge rests on capping slabs supported by columns which act as piers and abutments.

These columns are nothing but, the ordinary precast R. C. C. pipes that are available in the market. These R. C. C. pipes are so arranged with suitable concrete plugging and Reinforcement inside that they, as it were, act as piers for the Bridge superstructure to rest on.

The columns are composed of R. C. C. pipes of different diameters assembled with suitable overlaps. For the Foot Bridge illustrated, foundations were taken to a depth of 8'-0" below the Mean Bed Level. The lowest 6'-0" of the columns is of 3'-0" diameter R. C. pipes.

The Reinforcement consists of 4 Nos. $\frac{3}{4}$ " diameter bars with stirrups of $\frac{3}{8}$ " diameter at 12" c/c. carried to the capping slab above, right from the very bottom of the pipe. The hollow space inside the pipe is thoroughly plugged up, with concrete of correct proportion 1 : 2 : 4. The top two feet is left unplugged with concrete and 2'-3" diameter R. C. pipe is inserted into it, so that it remains upright overlapping the bottom pipe to a depth of 2'-0". Then the same process of concrete plugging

is continued up to its top two feet when 1'-6" diameter R. C. pipe is inserted with an overlap of 2'-0". It is again plugged with concrete upto its full height, the Reinforcements in it being carried a few inches beyond the pipe which affords a rigid connection of the column with the capping slab. The points of overlaps of the different diameter pipes are given conical concrete cappings as illustrated in the plan.

As regards the capping slab it is designed just as a Beam resting on two supports. In this type of Foot Bridge the masonry abutments and wingwalls do not carry any Load directly imposed on them, but they merely serve the purpose of a Retaining wall for the bank and a Revetment against erosion.

COST ANALYSIS

1. Total Cost of Bridge	...	Rs. 6,200	0
2. Cost of Bridge/Rft. of Roadway	...	Rs. 775	0
3. Cost of Bridge/sq. ft. of Roadway...	Rs.	33	12

It is evident that if the piers were of Masonry, the Foot Bridge could not have been constructed at such a low cost.

ADVANTAGES

The main aspect which greatly recommends this type of construction, is the economy resulted by the choice of the R. C. columns with the encasing R. C. Pipes. The use of R. C. pipes not only effects economy but also affords facilities for easy construction especially when there is water in the stream. Further it serves the purpose of mould also, besides enabling the bearing area of the column to be increased. Thus the use of R. C. pipes has three-fold of objects *viz.* :

1. It makes it possible to lay the concrete for the columns even when there is water in the stream.
2. It serves the purpose of a mould.
3. It enables the bearing area of column to be increased so as to distribute the load uniformly on the soil, keeping the soil pressure within permissible limit.

Further since the masonry abutments and wing walls do not carry any load directly imposed on them, they have to be designed only to resist the Earth-pressure of the soil retained consequently their foundation depth and size can be limited to the actual requirements which further adds to the economy.

In view of these considerations both of economy and of easy construction, this type of Foot Bridge commends itself to be used with advantage in small village Roads where a Bridge of this type is meant only for the Vehicular Traffic.

DESIGN

1. **Deck Slab.**—This is designed as a simply supported slab.

For the purpose of computation of its self weight, assume a tentative thickness of 9" for the slab.

Self weight = $9 \times 11.5 \times \frac{9}{12} \times 150$ lbs. since span is 10' - 0" clear, width of slab is 9' - 0" including kerbs, and length is 11' - 6" between centre to centre of columns.

∴ Self weight of = 11644 lbs.

Taking a maximum live load of 3 tons.

Super load = $3 \times 2240 = 6720$ lbs.

Weight of wearing coat 3" thick.

$$= 1 \times 11.5 \times 9 \times \frac{1}{12} \times 150 = 3881 \text{ lbs.}$$

Add for weight of kerb, and hand rails

$$= 2755 \text{ lbs. (Lump sum)}$$

Total load = 25000 lbs.

Effective span = 11' - 6"

Considering 12" width of slab in a direction perpendicular to the direction of the Traffic and assuming the total load is uniformly distributed over the entire width of slab,

$$\begin{aligned} \text{Load on the 12" strip} &= \frac{25000}{9} \\ &= 2777.7 \text{ lbs. (or) } 2800 \text{ lbs. say.} \end{aligned}$$

Assuming that the slab is simply supported, we have maximum bending moment

$$= \frac{2800 \times 11.5 \times 12}{9} \text{ in lbs.}$$

$$= 125 \times 12 \times d^2$$

where 'd' is the effective depth of slab required.

$$\text{Hence } d = \sqrt{\frac{2800 \times 11.5 \times 12}{8 \times 125 \times 12}} = 5.65"$$

Allowing a cover of 2.35", we have total depth D = 8"

Reinforcements :

Area of steel required to resist the Max^m B.M. is given by $A_s f_s Jd = \text{Max^m B.M.}$

R. C. Foot Bridge

Assuming $f_c = 750$ psi
 $f_s = 18000$ psi
 $J = 0.9$
 $m = 15$, we have

$$A_s \times 18000 \times 0.9 \times 5.65 = \frac{2800 \times 11.5 \times 12}{8}$$

$$\therefore A_s = 0.5330 \text{ sq. in.}$$

Using $\frac{3}{4}$ " ϕ bars,

$$\text{Spacing} = \frac{12 \times 0.4418}{0.5330} = 9\frac{1}{2}"$$

Hence $\frac{3}{4}$ " ϕ bars @ $9\frac{1}{2}"$ c/c will suffice.

Distributors :

Using 20% of Main Steel area, $a_s = \frac{0.5330}{5} = 0.1066 \text{ sq. in.}$

∴ $\frac{3}{8}$ " ϕ bars @ 12" c/c can be used as distributors.

2. **Capping Slab.**—This is designed as a simply supported Beam.

Effective span = 6' - 0".

Assuming a tentative size of 24" $\times$ 12" for the Beam.

$$\text{Self weight} = 2 \times 1 \times 9 - 0 \times 150 = 2700 \text{ lbs.}$$

$$\text{Super load} = 3 \times 2240 = 6720 \text{ lbs.}$$

$$\begin{aligned} \text{Weight of deck slab} &= 9 \times 11.5 \times \frac{1}{12} \times 150 \\ &= 10350 \text{ lbs.} \end{aligned}$$

Add for weight of wearing coat 3" thick

$$= 1 \times 11.5 \times \frac{1}{4} \times 150 = 3881 \text{ lbs.}$$

Weight of Hand Rails and kerb = 2755 lbs.

(lump sum)

$$\text{Total load} = 26406 \text{ lbs.}$$

$$= 26400 \text{ lbs. (say)}$$

Maximum bending Moment = $\frac{26400 \times 6 \times 12}{8}$ in lbs.

$$\therefore 126 \times 24 \times d^2 = \frac{26400 \times 6 \times 12}{8}$$

Where 'd' is the Eff. depth of Beam.

$$\therefore d = \sqrt{\frac{26400 \times 6 \times 12}{8 \times 126 \times 24}} = \frac{8.86}{9.77} \text{ (or) say}$$

Adding a cover of 3",

Over all depth D=12".

Hence a section of 24" $\times$ 12" for the capping slab will suffice.

Reinforcement :

$$As \times 18000 \times 0.9 \times 8.86 = \frac{26400 \times 6 \times 12}{8}$$

$$\therefore \underline{As = 1.332 \text{ } \square''}.$$

$\therefore \frac{3}{4}'' \phi$ Main bars 4 Nos. are more than sufficient.

Stirrups :

Using 20% of Main Steel area, as = $\frac{1.332}{5} = \underline{0.2664 \text{ } \square''}$

$\therefore$ Using $\frac{1}{2}'' \phi$ Stirrups,

$$\text{Spacing} = \frac{12 \times 0.1963}{0.2664} = \underline{8.75'' \text{ c/c say.}}$$

$\therefore \frac{1}{2}'' \phi$ Stirrups @ 8.75" c/c will suffice.

3. **Columns.**—The column consists of three different diameter R. C. pipes and consequently the area of cross section is not uniform. Hence the properties of such a column is entirely different from that of a column of uniform cross section. Consequently the ordinary column formulae may not apply to this case. But it is easy to realise that we will be erring only on the right side by applying the usual formulae in this case.

The assumptions to be made in this case are :

1. The column is short.
2. Both ends of the column are fixed rigidly in position and direction.
3. The properties of the column are equivalent to that of a column of average cross sectional area.
4. The properties of the column are not affected by the encasing R. C. pipes.

Safe axial load on a short column with longitudinal bars and lateral ties is given by,

$$P = f_c A + f_s As.$$

Where f_c = Permissible stress in concrete in direct compression = 600 lbs./ $\square''$ for 1 : 2 : 4 mix (*vide* clause 308 Madras Highways Manual).

A = Area of cross section excluding area of Reinforcement.

f_s = Permissible compressive stress in steel = 13500 lbs./ $\square''$ (*Vide* clause 309 Madras Highways Manual).

and As = Area of longitudinal bars.

Average column Diameter = 2' - 3" Reinforcement consists of 4 Nos. $\frac{3}{4}'' \phi$ Bars and $\frac{3}{8}'' \phi$ stirrups @ 12" c/c. In the present case, load on each column = 13200 lbs.

$$A = \frac{\pi (27)^2}{4} - 4 \times 0.4418 = 566.03 \text{ } \square''$$

$$As = 4 \times 0.4418 = 1.7672 \text{ } \square''$$

Hence $P = 600 \times 566.03 + 13500 \times 1.7672$
 which is evidently > 13200 lbs.
 Hence the section is safe.

Check for Soil Pressure :

Total load on each column = 13200 lbs.
 Self weight of column, taking average section
 $= \pi \frac{(2.25)^2}{4} \times 14 \times 150 = 8353$ lbs.
 Since total height of column is $14' - 0''$.
 Add for weight of R. C. pipes = 847 lbs. (L.S.)
 Total load on soil = 22400 lbs
 (or) 10 tons.

The column is held in Equilibrium partly by the frictional resistance of the soil surrounding the pipe and partly by the bearing underneath.

Considering only the bearing.

$$\text{Pressure on soil} = \frac{10}{\pi (1.5)^2} = 1.414 \text{ tons}/\square'$$

The soil met with is stiff clay whose bearing capacity may be taken as 2 tons/ $\square'$

Here pressure on soil < 2 tons/ $\square'$

Hence it is safe.

Check for Frictional Resistance :

Mean bed level = 39.00.

Foundation level = 33.00.

$\therefore$ Depth of column below bed level = $6' - 0''$.
 Peripheral area in contact with soil = $\pi \times 3 \times 6$
 $= 56.57 \square'$

Assuming that all the Load is sustained by the frictional resistance, we get Stress

$$\begin{aligned} \text{developed} &= \frac{10 \times 2240}{56.57} \\ &= 3951 \text{ bs.}/\square' \end{aligned}$$

This is well below the allowable stress of 800 lbs./ $\square'$.

Hence Safe.

In this case, no check need be made for punching shear since the column is continuous and monolithic and the R. C. pipes merely encases the concrete. In Fact, the purpose of R. C. C. pipes is threefold, viz. : to enable the concrete to be laid even when there is water in the stream, to serve as a mould and to be laid even when there is water in the stream, to serve as a mould and to increase the Bearing area.

Hence one R. C. pipe overlapping another does not produce any punching shear which follows *a fortiori* from the fact that the monolithic column only takes the load but not the R. C. pipes.

4. **Abutment and Wing Walls.**—These carry no load from the Bridge super structure but it merely protects the approach against erosion.

Hence the loading to be considered in the design are only the pressure of Earth-fill Retained and its self weight. The base width required in this case can easily be computed by the ordinary Rankine's Formula.

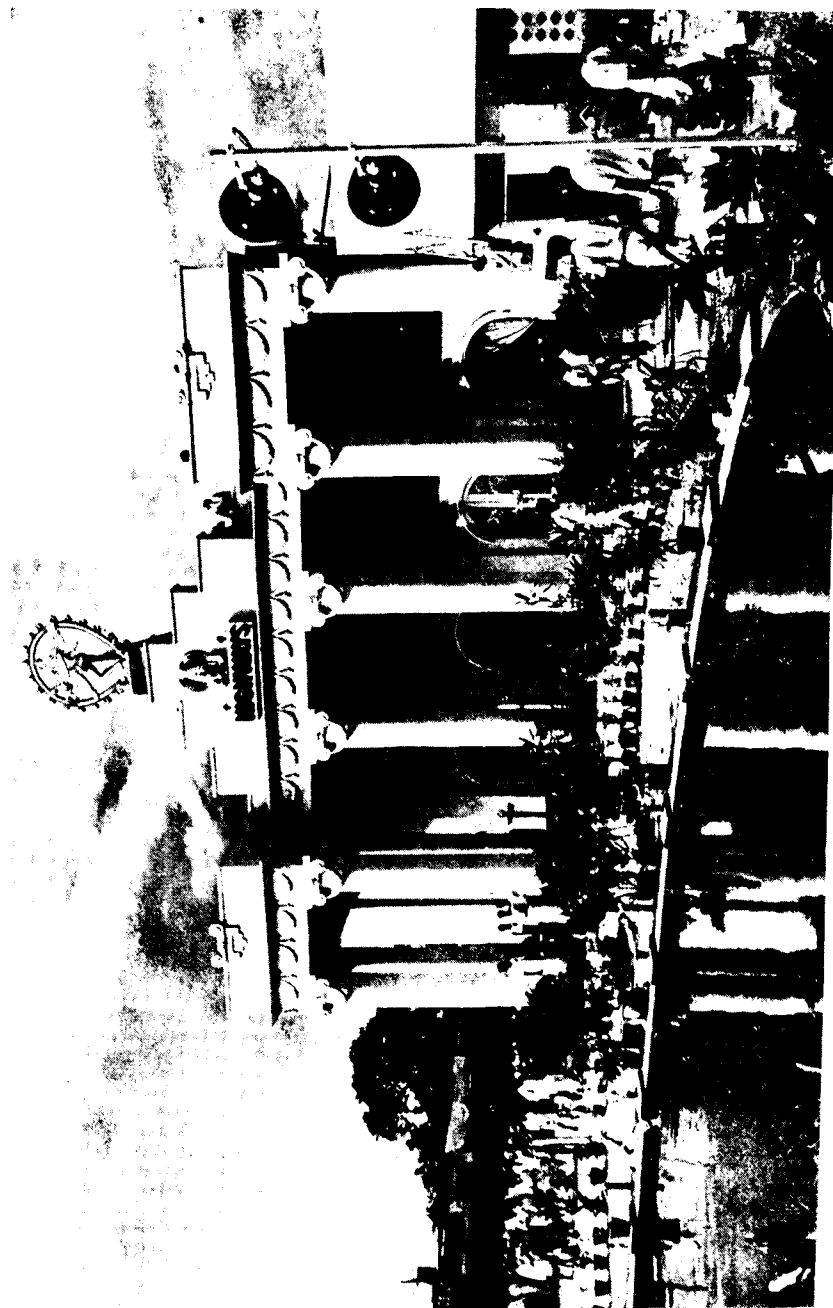
A base width of $4' - 6''$ will be found to be sufficient for Masonry and giving an offset of $6''$ on either side, a width of $5' - 6''$ may be used for the concrete footing.

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Centenary Exhibition

Centenary of the Madras University was celebrated with great eclat befitting the important occasion. One of the high lights of the celebration besides special convocation for conferring honorary doctorates on important personages of India and abroad and the laying of foundation stone of the University Centenary buildings by the Prime Minister was the Centenary exhibition which can be said to be the biggest ever organized scientific exhibition in South India. Almost all the important educational institutions and Government Departments like Medical Colleges, Engineering College, Alagappa Chettiar College of Technology, Queen Mary's College, Public Works Department, Electricity Department etc., etc. participated and our department spared no pains in putting up a really interesting and instructive show. From the visitor's book kept in the stall of Highways Department could be seen the great appreciation, of eminent men and women of our country and the foreign people who were on a visit for the exhibits put up by us. The University authorities awarded 'A' class shield to our department in appreciation of the elegance with which the exhibits were displayed. Our congratulations go to the officers and subordinates who spared no pains in contributing for the success under the guidance of our Chief Engineer (Highways).

The photographs given below show a few of the most interesting exhibits displayed by us.



The entrance facade of Exhibition Stall of the Highways Department



The Exhibition was opened by Sri C. D. Deshmukh, Sri K. K. Nambiar, Chief Engineer is explaining the model of a bridge construction to the distinguished visitors



The working model of a design for a traffic interchange—this attracted the attention of young and old alike.



The working model of a lift bridge. Note the steamer moving in after the bridge deck having been lifted.



Sri K. K. Nambiar, Chief Engineer (H) receiving the shield from Sri P. V. Rajamannar, Chief Justice, Madras

V-SHAPED FRAMES FOR SUPPORTING BRIDGES

[This article is a translation of a paper published in "Acta Technica", the journal of the Academiae Scientiarum Hungaricae, of Budapest, by permission of the Academy.]

In *Fig. 1* are shown examples of bridges with inclined columns compared with bridges supported by vertical columns; at (a) is shown to the left a bridge with a beam extending to form cantilevers over two columns, and to the right is the proposed construction with inclined columns; at (b) a three-pinned arch is compared with a statically-determinate application of the proposed method; and at (c) are alternative methods of construction of a three-bay frame. These examples show some of the applications of the method in which vertical supports subjected to bending and compression are replaced by inclined supports subjected to direct tension or compression only. The analysis is limited to a single-span bridge with cantilevers such as that shown in *Fig. 1 (a)*. This form of structure is often used for bridges such as those carrying railways over ravines or roads over railway cuttings.

THEORY OF TWO-HINGED FRAMES ON V-SHAPED SUPPORTS

In principle the calculations are similar to those for the normal type of frame with columns subjected to bending. The only difference is that the columns are replaced by inclined members which resist the horizontal and vertical forces by direct compressions and

tensions rather than by their resistance to bending; both types of structure are statically-indeterminate in the first degree.

To simplify the calculation, assume that the beam is of constant moment of inertia and that the influence

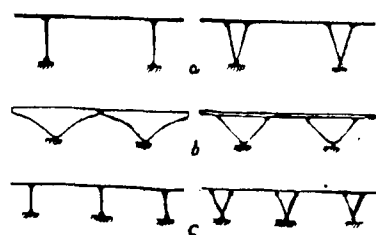


FIG. 1

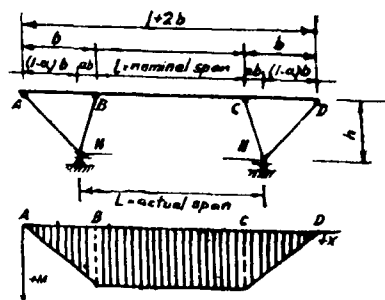


FIG. 2

of the axial deformations of the supports is negligible. Fig. 2 shows the primary system of the proposed construction, one of the bases being hinged and the other on rollers so that the statically-indeterminate force may be defined as the horizontal reaction H .

In order to draw the influence line for H , assume that unit horizontal forces act at the level of the hinges at the base. The bending moment on the beam due to

these forces is shown in Fig. 2. The influence line for H is determined by

$$\eta h = -\frac{a_{01}}{a_{11}} = -\left(\frac{a_{01}}{a_{11}}\right)$$

where a_{01} is the deflection due to bending of the beam and a_{11} is the horizontal deflection of the movable support due to the action of a unit force H . The values in parentheses are the product of the deformations and the flexural rigidity EI .

CALCULATION OF a_{01} .

The elastic line, or the deflection-curve, is easily determined from the second integral of bending moment due to the unit horizontal force

$$\frac{d^2y}{dx^2} = -\frac{M}{EI}$$

where y is the deflection, x the position of the section considered with respect to the origin, and M the bending moment. The expression is integrated in two portions, one for the part b of the beam between the inclined supports and one for the span l between the inner legs of the inclined supports.

In the length b :

$$M_b = +\frac{h}{b}x.$$

$$(\phi b) = -\int M_b dx = -\frac{h}{b^2}x^2 + c_1.$$

$$(y_b) = \int \int M_b dx dx = -\frac{h}{6b}x^3 + c_1x + c_3.$$

In the span l :

$$M_l = +h.$$

$$(\phi l) = -\int M_l dx = -hx + c_2.$$

$$(y_l) = -\int \int M_l dx dx = -\frac{hx^2}{2} + c_2x + c_4.$$

The constants of integration c_1 to c_4 are calculated from the following conditions.

(a) At the middle of the span l the tangent to the deflection-curve is horizontal if the structure is symmetrical:

$$[\phi l]_{x=b+\frac{l}{2}} = 0.$$

$$\text{Hence } c_2 = \frac{h}{2} (2b + l).$$

(b) At B the elastic curve is continuous, that is

$$[\phi l]_{x=b} = [\phi b]_{x=b}.$$

$$\text{Consequently } c_1 = \frac{h}{2} (b + l).$$

(c) There is no vertical displacement of the intersection of a straight line joining the points of support A and B and a vertical line passing through the hinge at the base. Also, the effect of axial deformation of the inclined supports is neglected. Given these conditions the horizontal movements of A and B have to be in an opposing sense and their values are proportional to their horizontal distances from the hinges at the base, thus

$$\frac{[\gamma b]_{x=0}}{(1-\alpha)b} = -\frac{[\gamma b]_{x=b}}{\alpha b}.$$

Consequently

$$c_3 = -(1-\alpha) \frac{hb}{6} (2b + 3l).$$

(d) At B the curve of deflection is continuous, hence

$$[\gamma l]_{x=b} = [\gamma b]_{x=b}.$$

Consequently,

$$c_4 = \frac{hb^2}{6} - (1-\alpha) \frac{hb}{6} (2b + 3l).$$

With these values for the constants of integration the equations of deformation become

$$(\gamma b = (a_{01})b = -\frac{h}{6b} [x^3 - 3b(b+l)x + (1-\alpha)b^2(2b+3l)],$$

$$(\gamma l = (a_{01})l = -\frac{h}{6} [3x^2 - 3(2b+l)x + (1-\alpha)b(2b+3l) + b^2].$$

CALCULATION OF a_{11} .

According to the relations determined by the bending moment diagram in *Fig. 2*,

$$(a_{11}) = \frac{h^2}{3} (2b + 3l).$$

Consequently the equations of the influence lines for the two sections are

$$\eta b = \frac{x^3 - 3b(b+l)x + (1-\alpha)b^2(2b+3l)}{2bh(2b+3l)},$$

$$\eta l = \frac{3x^2 - 3(2b+l)x + (1-\alpha)b(2b+3l) + b^2}{2h(2b+3l)}.$$

These equations determine the influence line for the horizontal force H from which may be determined the influence lines for the internal forces, bending moments, and shearing forces, and hence the stresses at any point in the structure may be calculated. By this method of calculation it is simple to determine the stresses in any statically-indeterminate structure supported on inclined columns. As previously mentioned, the shortening or lengthening of the inclined supports is neglected as the effect on the forces in the frame is generally negligible; nevertheless, these deformations may easily be accounted for in the calculations if this is considered necessary.

STATICAL ANALYSIS

In the design of a structure with inclined columns there is a much greater freedom of choice of dimensions

than there is with the usual method of construction in which, for example, the choice is limited to determining the spans from an overall length of a bridge. Because of this, the analysis of frames with inclined columns is limited to the single case of a two-pinned frame with cantilevers in which the possible variations include the lengths of the cantilevers, the distance between the inclined members and hence the nominal span, and the real span of the frame which is the distance between the hinges at the base. Three cases are shown in Fig. 3, in which the nominal span is assumed to be unity and the over-all length of the frame 2.5 units. The equations for the influence lines for H have been calculated for these cases.

Fig. 3a : The outer supports are vertical and $x = 1$:

$$\eta b = 0.5x^3 - 1.125x.$$

$$\eta l = 0.75x^2 - 1.5x + 0.065.$$

Fig. 3b : $x = 0.5$ where the supports form equilateral triangles :

$$\eta b = 0.5x^3 - 1.125x + 0.25.$$

$$\eta l = 0.75x^2 - 1.5x + 0.3125.$$

Fig. 3c : $x = 0$ and the interior supports are vertical :

$$\eta b = 0.5x^3 - 1.125x + 0.5.$$

$$\eta l = 0.75x^2 - 1.5x + 0.5625.$$

The influence lines for the three cases are the same except that the base-line is varied as shown in Fig. 4. It may therefore be seen that by altering the real span

of the frame the positive and negative parts of the bending-moment diagram may be altered to give the most favourable values. For example, in the case of a frame in which the dead load (assumed to be uniformly distributed) is preponderant the positions of

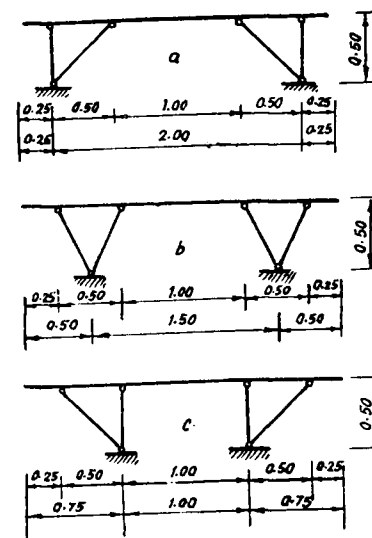


FIG. 3

the hinges at the bases would be chosen so that the total area of the bending-moment diagram approaches zero, and hence for the dead load the forces would be vertical only, that is H would be zero. It is frequently of importance with certain foundations that this should be so. The only horizontal force on the base would be that due to the imposed loads and would be of secondary importance in the design of the foundations.

Fig. 4 shows also the influence line for H for a three-pinned frame (shown in a chain-dotted line) and it may be seen that in general the horizontal thrust is

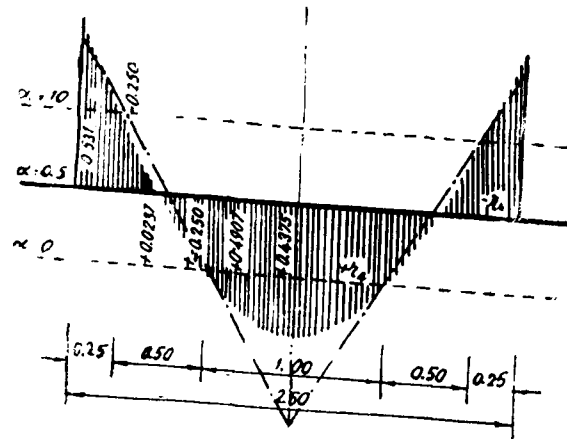


FIG. 4

greater for a three-pinned frame than for a two-pinned frame.

COMPARISON OF BENDING MOMENTS

In Fig. 5 are shown two frames, one with vertical columns and one with inclined columns. For these frames the envelopes of maximum bending moments are shown in Fig. 6; the bending moments due to dead loads only are shown by a dotted line. The maximum bending moments are those due to dead and live loads, variation in temperature, and shrinkage of the concrete. Due to the rigidity of the inclined columns the effects of changes of temperature and of shrinkage are greater for the frame with these columns than for the frame with vertical columns. Comparing the bending moments in the two forms of construction, the maximum positive moment in the normal frame is 365 tonnes-metres against 265 tonnes-metres for a frame with inclined columns; similarly the comparative negative bending moments are 579 tonnes-metres and 288 tonnes-metres. That is, the positive bending moments have

been reduced by 27 per cent, and the negative bending moments by 50 per cent., while the total area of the bending moment envelope has been reduced by 40 per cent. Such reductions have a considerable influence on the sizes of the members and consequently on the dead load, but the bending moments given do not take this into account as both cases have been calculated for the same loads.

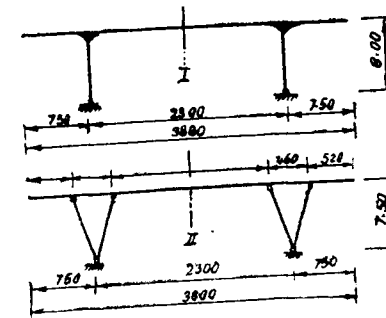


FIG. 5

The shearing forces are also reduced, particularly in the part of the beam between the inclined supports where the shearing force is greatest in the general case.

DISPLACEMENT OF THE SUPPORTS, AND DEFLECTIONS

Neither the frame with vertical columns nor the frame with V-shaped supports is sensitive to vertical movements, but horizontal displacement has a greater effect on the frame with V-shaped supports than on the normal frame.

Deflections of the V-frame are less than those of the normal frame, as may be seen from Fig. 5. In bridges this is important, particularly in relation to the joint between the ends of the cantilevers and the road

slab. Frequently the length of a cantilever is less than about 24 ft. as otherwise the deflections under a moving load may damage the surface. In the construction

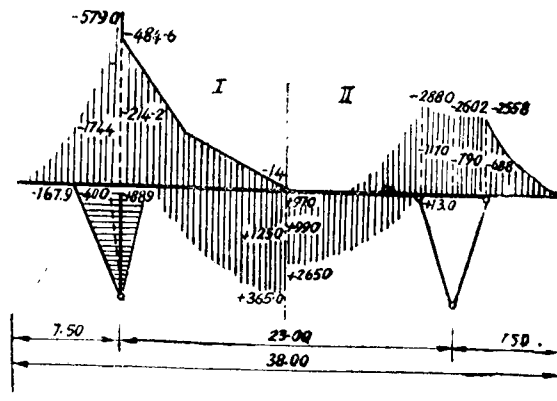


FIG. 6

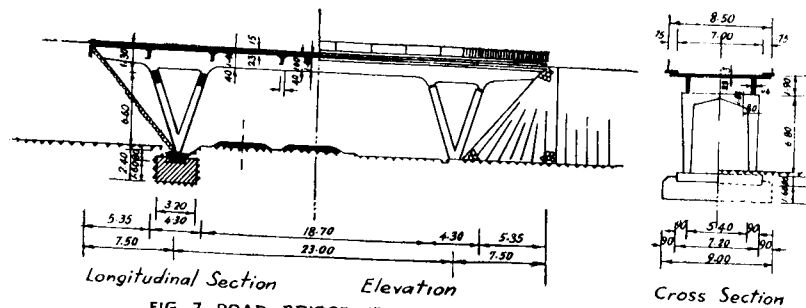


FIG. 7 ROAD BRIDGE WITH SUPPORTS HINGED AT MAIN GATES

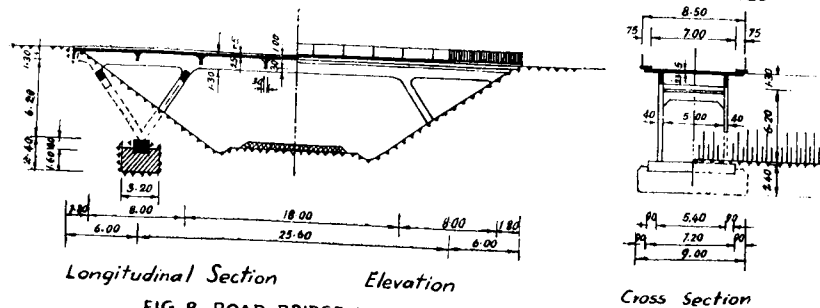


FIG. 8 ROAD BRIDGE WITH SUPPORTS RIGIDLY CONNECTED TO MAIN BEAMS

proposed in Fig. 5, for example, it is possible to reduce the length of the cantilever from 24 ft. to 16 ft., and as the deflection is proportional to the cube of the span the deflection in this case is reduced by one-third.

EXAMPLES OF BRIDGES WITH V-SHAPED SUPPORTS

Fig. 7 Shows a bridge carrying a road over a rail way cutting. As in the case of the frame for which the bending moments have been calculated, this bridge consists of two main beams at about 18-ft. centres carrying transverse beams at 16-ft. centres; the deck is a slab spanning in two directions and cantilevering over the main beams to form footpaths. The main beams are supported on inclined columns forming three-pinned frames across the width of the bridge. This arrangement was adopted to resist lateral wind pressure.

A similar bridge is shown in Fig. 8 but here the greater part of the inclined columns is buried in the embankment. The two bridges are the same in principle, the difference being that in Fig. 7 the supports are hinged at their intersection with the main beams while in Fig. 8 the supports are continuous with the main beams. In the latter case bending moments are produced in the inclined columns but, because of their flexibility compared with the beams, the bending moments are small (they can easily be determined by the moment-distribution method).

ARCHITECTURAL ASPECTS

For structures such as these it is not only necessary to determine the structural advantages of using

V-shaped supports but also to consider the appearance of the structures. Elevations are shown in *Figs. 9 and 10* of the two bridges of which the structural details are given in *Figs. 7 and 8*. It is not assumed that these sketches show the best aesthetic development of the proposed method, but they compare favourably with other methods.

Fig. 11 shows a road-bridge in Hungary. The bridge carriageway 22 ft. wide and two foot-paths each about 2 ft. 6 in. wide over a railway cutting. The angle of intersection of the road and the railway is about 60 deg. The bridge comprises two main beams, each about 4 ft. 3 in. deep at the centre, supported on inclined columns hinged at both top and bottom. The total length of the bridge is 132 ft., the cantilevers are about 28 ft. 9 in., and the actual span of the frame is 74 ft. 6 in. The length of the cantilevers was chosen so as to reduce as much as possible the horizontal forces at the bases due to the dead loads.



FIG. 9

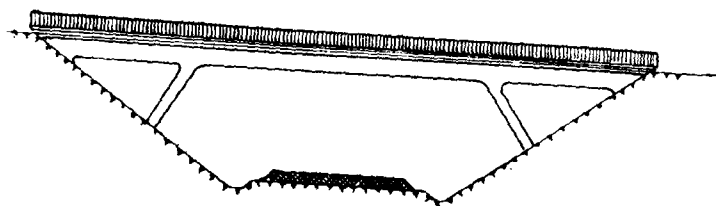


FIG. 10



Fig. 11. - Bridge over a Railway.

The quantities of materials used were—concrete : 213 cu. yd. (0.52 cu. yd. per square yard) ; shuttering : 990 sq. yd. (2.45 sq. yd. per square yard) ; steel : 33 tons (182 lb. per square yard). Quantities of materials calculated for this bridge with vertical columns where about 10 per cent. greater.

—(Extracted from "Concrete & Constructional Engineering".)

BRIDGES AND RIVER CONTROL-USE OF HYDRAULIC MODELS

Rivers constitute one of the most important natural resources of a country and occupy a very prominent place in every stage of human development.

In olden times, when railways and highways were not developed, rivers were the chief means of communication for inland transport. But with the development of our new modes of communication, rivers offer an obstacle that needs to be bridged over.

Bridge-building, though an ancient art, developed rapidly in India as elsewhere in the world during the last century along with the expansion of our roads and railways. Methods of design varied according to the type of river to be bridged.

Broadly speaking, rivers in the Deccan which flow through deep channels with stable banks and have sound rock foundations, offer no special difficulties about the safety of the structures. But it is the rivers in the Indo-Gangetic and other flood plains that require special considerations to be given for achieving economy in the cost of construction and also ensuring the safety of the bridge and subsidiary structures.

LARGE WIDTHS

These rivers flow through the deep alluvial beds which are unstable. Their channel widths are very large, though only a small fraction is occupied by the cold weather flowing channel but in monsoon when they are in spate, full section is occupied though a substantial part of it, is fairly shallow and carries only

sluggish flow. Also due to unstable bed the position of the main channel carrying bulk of the discharge keeps on changing.

Excessive cost of construction of unusually long bridges covering the width of the river from bank and the difficulty of protecting them and their approaches from the ever wandering courses of these rivers formed a major problem confronting the bridge engineer. Solution was suggested by the observation in nature of a wide river passing often through a stable narrow gorge with a much narrower but deeper section.

NARROWING THE RIVER

Artificial narrowing of the river in conformity with such observations has been found to be possible and economical. It has a further advantage of controlling the river by preventing its swings over a wide region. Such artificial narrowing is achieved by constructing heavy embankments called guide banks near the abutments and closing the remaining portion of the river by approach banks. The main idea is that the river flow should approach and pass through the bridge smoothly.

With this object in view of the different aspects of the bridge design that need consideration areas follows:

(1) Selection of site; (2) determination of waterway; (3) design of guide banks, approach banks etc. and (4) bridge piers.

It is for the study of these points that hydraulic models place before the designer a vivid picture of what the state of affairs is going to be after the bridge is constructed. All these aspects can be studied at a

negligible cost, so as to achieve economy consistent with stability and strength in the initial outlay and also reduce maintenance.

IDEAL BRIDGE SITE

An ideal bridge site should in a narrow straight and stable reach of the river. The orientation should be such as to have a normal flow through it.

But conditions in nature are generally not ideal, and here the model helps the engineer to see for himself the alignment that approaches the ideal. Hence a site is first roughly selected by considering the layout of the river channels, changes in the meander positions in the past, proximity of a nodal region, availability of a rigid bank, existence of a narrow meander belt and the river bed material and then tested on the model. After examining the details of currents on the model the exact site and the orientation can be easily fixed.

In the case of the proposed bridge at Mokameh Ghat similar model studies were carried out at the Central Water and power Research Station, Poona. The bridge alignment was confirmed as the flow conditions were found to be quite satisfactory.

Extract from "The Auto-Spark", February 1956.

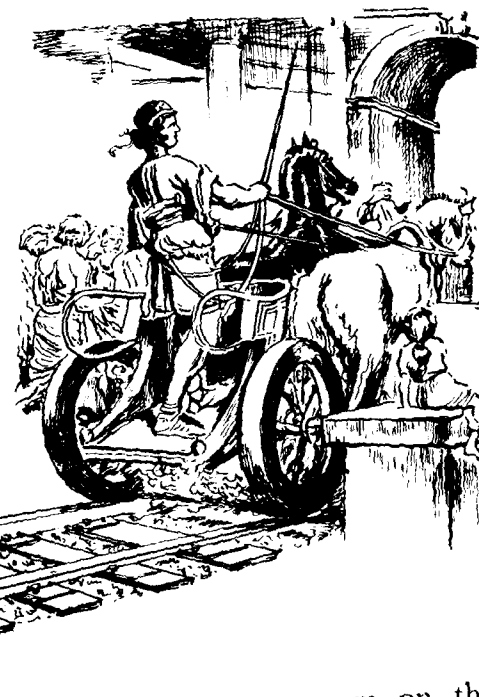
ITEMS of interest

THE TRACKS OF HISTORY

With the exception of Russia practically the entire civilized world now uses the standard railroad gauge of 4 feet 8½ inches.

There is unexpected historical romance behind the adoption of this gauge.

This width was used by the Roman chariots with which Julius Caesar's army invaded Britain in 55 B. C. The ancient chariots were copied by the native Celts of the British Isles with the result that



deep ruts 4 feet 8½ inches wide were worn on the English roads in the course of the centuries. Vehicles were made just wide enough to follow in the ruts.

Today more than 2,000 years after Caesar's invasion, the modern railway literally rolls on indelible tracks imprinted by the ancient Roman charioteer.

—From “Ripley's New Believe it or not”.

* * *

The Standard gauge of Great Britain, 4 ft. 8½ in. is also the normal gauge of Norway, Sweden, Germany, Austria, and central Europe generally, and is much used in France. Russia and Finland have a 5 ft. gauge.

The Standard Russian gauge of 5 ft. is said to have been chosen in order to prevent German rolling stock being serviceable in Russia. The principal Indian gauge is 5 ft. 6 in. which will not permit Russian wagons to run in India in the event of Central Asian lines being extended to the Indian frontier.

—Extracted from “The principles of Economic Geography”, Dr. R. N. Rudmose Brown.

* * *

McADAM

John London McAdam, was perhaps the greatest Highway Engineer who ever lived and yet about whom much was not common knowledge. Elsewhere is given a brief life sketch and achievements of this pioneer in road construction. He revolutionised the technique of Highway construction and his name became synonymous with the scientific way of road building. He was the first to recognize the modern concept in road design that the road structure is as strong as its foundations.

Highway Engineers and officials throughout the country will do well to pause on September 23 and pay tribute to the memory of the most famous practitioner of their art, John London McAdam. It is suggested an

“annual thanksgiving day” may be held to correspond with this date and all of us may dedicate ourselves every year to the ideal set out by John London McAdam.

Known as the “Colossus of Roads”. McAdam was Great Britain's premier turnpike trust administrator and engineer between 1816 and his death in 1836. Even as these dates almost exactly parallel the romantic coaching era, so did the man epitomize the spirit of the two decades between Waterloo and Queen Victoria's ascent to the throne.

John London McAdam was born in 1756 in Ayr, a country seat in the Southwest of Scotland, three years earlier than the poet Robert Burns, who was born in the same town.

On the death of his father in 1770, the 14 years old boy was sent to a wealthy merchant-uncle in New York City. There he witnessed the stirring scenes of the American Revolution, and was a leading merchant on the New York waterfront. He served as agent for the sale of at least 452 captured “rebel” vessels.

Persona non Grata when General Washington occupied the city and the independence of the colonies was recognized, McAdam and thousands of other Loyalists temporarily turned the tide of emigration to the New World. In June 1783 he landed at Ayr, and never again returned to North America.

By 1798 McAdam and his growing family were located at Falmouth, England, where he resumed his American occupation of prizemaster. But the Napoleonic

wars were interrupted by the abortive Peace of Amiens (1802-1803), and McAdam moved to Bristol. There he became a prominent merchant, and as a member of the Bristol Turnpike Trust he grew interested in the twin.

McAdam wrote four little volumes on turnpike administration and on engineering methodology, *One of these, Remarks on the present system of road making*, went through nine editions in England, one in the United States, and was translated into German. He advised the Post office, testified before committees of Parliament, and with his sons and other relatives created a national network of turnpike trust specialists. His son James, General-Surveyor of the combined Metropolis Turnpike Trust, was elevated to a baronetcy in 1834—as honour, it is said, his father refused.

Full of honors at the age of 80, McAdam died in 1836. Newspapers and periodicals throughout the British Empire noted his passing with regret. His name had already become a part of his native language. Almost all his contemporaries recognized his great contributions to the progress of civilization.

And the *Oxford English Dictionary* cites no less than six derivations of his name. His followers were called “macadamizers” Roads were being surfaced with “macadam” or simply “mac”; his system of making roads was styled “macadamization”; and to make or repair a road in accordance with his system was to “macadamize”.

A few were so entirely captivated by the achievements of the amazing Scot that they could hardly

restrain their active imaginations. Such a passion possessed Nimrod, who wrote, “I consider Mr. McAdam as being, next to Dr. Jenner, the greatest contributor to the welfare of mankind that his country has ever produced”.

* * *

ROADS, PAST AND PRESENT

The oldest major road in the world, in use centuries before the birth of Christ, crossed southwestern Asia to the Mediterranean Sea. It spanned a distance of nearly 1,750 miles. Hittite documents describing it go back to 2,000 B. C.

This ancient thoroughfare was named the Royal Road by the Persians. It has measured the strides of the great ancient civilizations. The Babylonians, Assyrians, Chaldeans, Cretans, Hittites, Phoenicians, Lydians, Phrygians, and the Medes are only distant references in history books to us. Yet these peoples contributed largely to our present-day society in transmitting the discoveries of their age along the Royal Road and other great highways.

The Egyptians, Greeks, Persians and Romans took their knowledge and passed it on across this route to the West. Through the turmoil and thousands of years of these civilizations, the road remained a constant connection between the East and the West. The civilization disappeared and finally, about 300 B. C., the road perished.

Travellers in those faraway days could leave Susa, located in the vicinity of what is now the south Iran-Iraq border and go north to Nineveh on the Tigris River, the modern-day site of Mosul, oil center of Iraq.

An ancient inscription, left by the Assyrian King of Nineveh between 705-681 B. C. says: "Royal Road. Let no man decrease it". The Royal Road which passed through this ancient city was about 78 feet wide. Punishment by death was the rule for any person who encroached upon its width.

The road crossed the flaming deserts to the ancient city of Samosata on the Euphrates, the starting point of the silk route to India, and then proceeded in two sections to what is now Kayseri in modern Turkey.

From here it went north towards the Hittite kingdoms on the Black Sea and westward to the Mediterranean. Supplementary routes extended around the arc of the eastern Mediterranean Sea to Cairo.

This route, now forgotten in the sands of time, brought to the West the wheel and axle, the domesticated horse, and the cultural richness of Asia Minor. Along its mighty length, a distance over half across the United States, the conquering armies of the ages marched.

To the Old Silk Road, mentioned in the first article of this series, however, goes the record for being the longest highway in all of history, including the present. It is also the oldest continuously-used road in the world. As early as the Second Century A. D., the thriving traffic along this route was an important contribution to the luxurious living of the wealthy classes of the Roman Empire. This great East-West highway extended from Peking and Shanghai in China to modern-day Cadiz in Spain. Its length girdles more than one-third the circumference of the globe—a total

of 8,000 miles. The terrain is tortuously difficult, ranging from below sea level desert lands to heights of 4,000 feet skirting the Asiatic Himalayan mountains. The Roman emperors rebuilt and established the route as an important part of the empire. In China, the road was known as the Imperial Highway and much of it was guarded by the fortification of the Great Wall.

During the days of the decline of the Roman Empire after the advent of the Third Century (101-200 A. D.) the Roman system of patrolling and maintaining the roads was gradually discontinued. They fell into disrepair everywhere. By the sixth Century, the narrow, selfish economic interests of local communities blotted out the free trade of the world. Barriers were erected to isolate cities one from another. Highways were torn up to prevent their use by military marauders. The fear of pillage and sack turned the clock backward for the people of the Dark ages. The only travel was local. The carriages of the Romans, Carthaginians and Egyptians were useless in a society without adequate roads. For nearly a thousand years, until long after the dawn of modern history, the world's highways suffered enormously.

However, man has to travel, and he has to have highways to do so. Even during the Dark ages, some notable roads were built. One such route was a Norse thoroughfare constructed during the Ninth Century called the Varangian.

It led from the Baltic Sea to the Black Sea, thence to Constantinople, travelling down the Volkov and Dneiper Rivers in what is now Russia. Kieve, in southern Russia, was an important city on the route,

the gateway to the North. This Russ-Norse commerce trial was an outstanding economic highway of medieval times.

The far-flung Arab empire also sustained travel, which all but stopped for centuries in western Europe. The Arabs built roads and kept them in repair for commerce. Interestingly enough religion played a big part in the highway-building program of the Arabs. In each Moslem lifetime, one visit must be made to Mecca. With Arabs in Spain and all of northern Africa, it became essential to them to maintain a roadway for the use of the faithful in their pilgrimage.

The Crusades also helped restore the highways. The increased travel stimulated by the Crusaders encouraged local areas to build roads and bridges again. But no plan was followed. Road were built in a helter-skelter manner and no system of maintenance was provided. A few toll roads also sprang up during the Middle Ages, none of them achieved any special distinction.

In the meantime, an engineering tragedy had fallen upon the Western World. The secret of making cement was lost. The Romans, building the great roads of the ancient world, had known it, but in the centuries of fighting and ignorance that followed, knowledge of how to make cement disappeared. Road construction was at a very primitive level during the war-torn centuries that followed.

Not until late in the 18th century was cement rediscovered, making possible hard surfacing for roadways.

While the so-called civilised nations of the world had virtually abandoned highway construction, it was reaching a high level in a remote part of the world—Central America.

The Mayan Civilization in Honduras and Yucatan, completely reclaimed by the jungle, resembles the architecture of the Babylonian Civilization. From evidences discovered in recent times it is possible to surmise that the Mayan cities were connected by hard-surfaced paved roads.

These roads, built about 500 A. D., were dug to bedrock, providing a solid foundation, and then filled in with broken lime-stone. Large stones were used at the bottom and fine stone toward the surface. Cement of lime and finely sifted white earth was spread over the roadway, making a hard, smooth surface.

This was in 500 A. D. when Roman highways were disappearing and the secret of cement lost on the European continent.

Many highways in the Mayan ruins of south Central America are 25 feet wide. Resting benches along the route were common, similar to those built by the Romans.

The Toltecs and the Aztecs also achieved great success as road builders. The Aztecs achieved a particularly remarkable feat, connecting "floating islands". The Valley of Mexico was once mostly lake. In the days of Montezuma, islands dotted the area, and upon these islands the cities and buildings of the Aztecs were built. They were connected by causeways erected from

the mainland and connecting the 'floating islands' (The Aztecs were guided by 'Divine' prophecy to build their city upon an island in the great lake once covering the area surrounding Mexico City).

Aztec roads were built up from crushed rock, earth and rubble deposited between parallel palisades. Surfacing was stone cobbles.

Extract from 'The Highway Magazine', June 1953.

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BRIDGE

In the beginning was the word, It descends from the Anglosaxon Brige or Hrieg meaning a ridge. To bridge means to be-ridge to erect an elevated structure over a depression.

From Bridges of Britain by D. E. Mare C. Eric.

ASSOCIATION

NEWS

Consequent on the formation of the new Mysore State, Sri G. S. Ganapathy, Superintending Engineer (Highways) Madras and President of our Association has been allotted to Mysore State and we are not fortunate to have Sri Ganapathy at our forthcoming annual gathering to conduct its deliberations.

It is needless to enumerate the genial qualities of Sri G. S. Ganapathy, as he has always been so very popular and well known among the Highway Engineers of this State. Sri G. S. Ganapathy endeared himself to one and all with his abounding sense of humour, his pleasing manners and sympathetic understanding of men. Our very best wishes will go to him for a fruitful and prosperous career in the new State of Mysore.

— 124058

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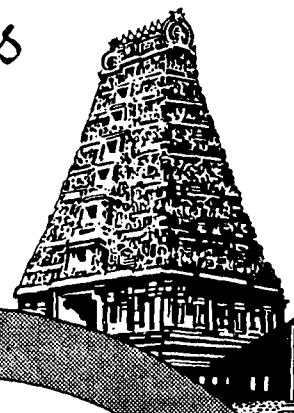


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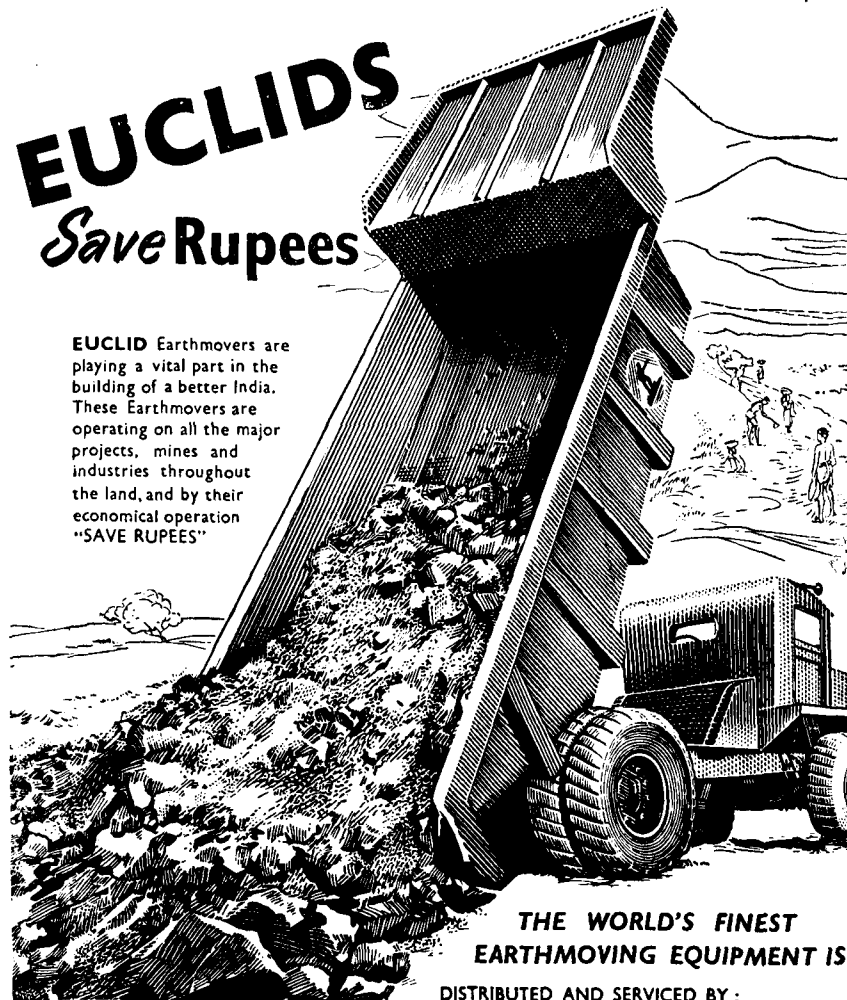
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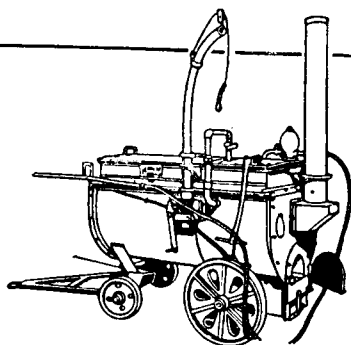


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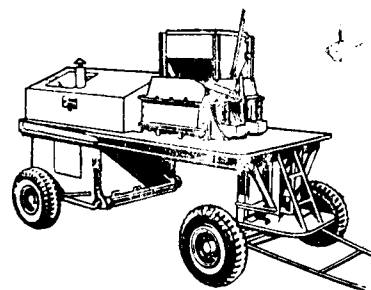
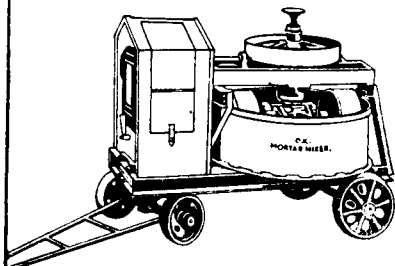
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