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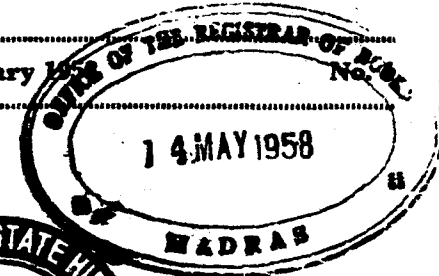
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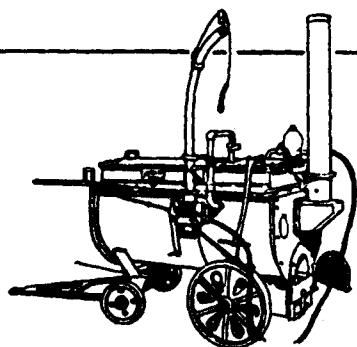
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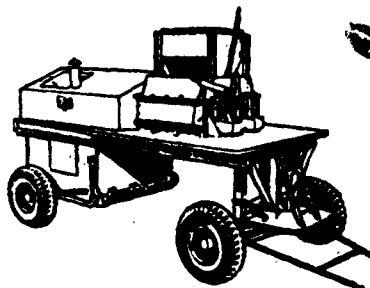
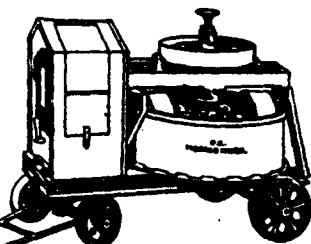


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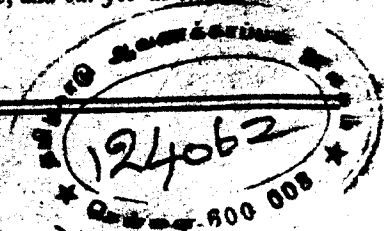
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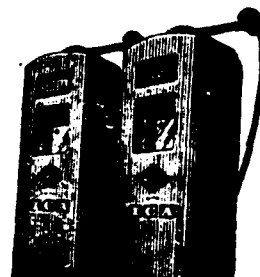
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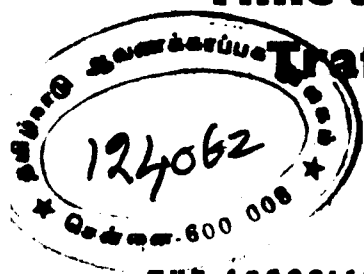


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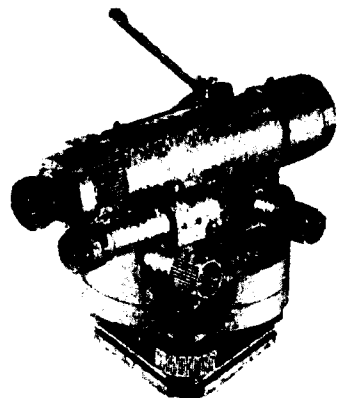
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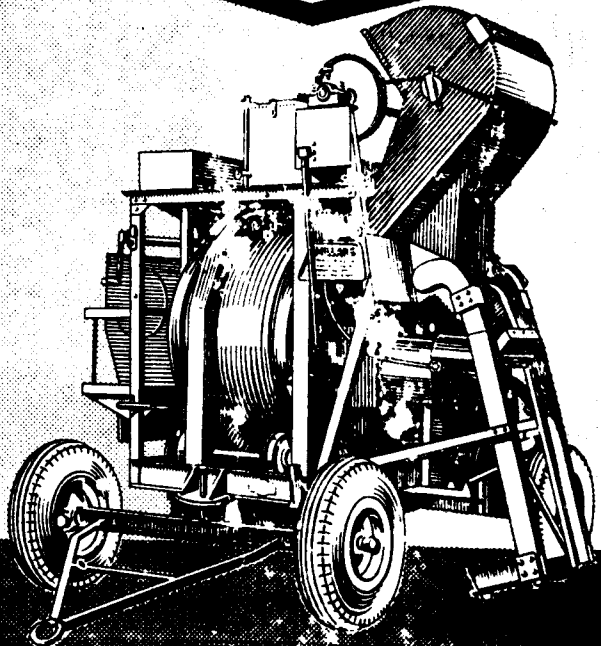
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MADRAS HIGHWAYS

THE JOURNAL

OF

THE ASSOCIATION OF
MADRAS STATE HIGHWAY ENGINEERS

Vo. III

January 1955

No. 1

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★

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It is well known that our present system of roads were evolved from ancient trails and cart tracks. It is by stage development that the present system of Highways has taken shape. Since the days when the motor vehicle first began to appear on our streets and highways, the problem of traffic safety has assumed great importance. The vehicles have been developed to extremely high capacities for performance and efficiency. But the element of highways which these vehicles perforce must use has not been quite safe and suitable for fast traffic. Hence there has been a great need for traffic control.

The traffic control will have to be two-fold restrictive measures and constructive measures. The aim of the former is to maintain public safety until such time as the roads have been made structurally safe. Constructive measures should be such that the vehicles track is well adopted and arranged for high speed without danger to public. Obviously the emphasis should be on engineering. Engineers, should not only have a responsibility of providing facilities but also for arranging for operation and efficient use of such facilities. While highway Engineers are applying themselves to improving highway geometry, width, sight distances, grades and curves, they have not been able to play an efficient role in enforcing restrictive measures mentioned above.

It is generally forgotten that the right of the public in a Highway is an easement of passage only—a right of passing and repassing. It is often seen that even when wide roads have been laid pushful owners of properties have gradually encroached and closed in on traffic width so that in and around inhabited areas what is called ribbon development have been taken place, which very much lessens the traffic capacity of such roads. Then there is the problem of nuisance committed on the road. Nuisance may be defined, with reference to highways as any wrongful act or omission upon or near a highway whereby the public are prevented from freely, safely and conveniently passing along a highway. The nuisance of most frequent occurrence are erections and excavations upon or adjoining highway. Again it is a common sight that cowdung, or paddy being dried on the highway, which can be classified as nuisance. Then there is the cattle straying on or lying on highway. These are but a few examples of the contributory causes for not having been able to make the best use of our highway. Extracted elsewhere is an interesting description of the conditions of our roads, by Mr. Hilton Brown, Retired I.C.S., which is forcibly expressive on the point at issue.

Therefore there is a great need for bringing about a comprehensive legislation, which provides facilities for the highway engineers to control the factors which contribute to making the highway less efficient. It may be of interest to mention that in United Kingdom first Highway Act came into existence in 1835. Ample powers were vested in the surveyor to deal with the matters connected with the highway law. It is timely that a comprehensive Highway Act is enacted for the conditions obtaining in this Country.

Improvements to Ponneri-Pulicat Road

BY

Sri G. KRISHNAN,

Assistant Engineer (Highways).

Introduction

The village Pulicat, situated along the East Coast about 20 miles North of the City of Madras, though at present contains a cluster of fishermen's huts, has got a history behind it. In the year 1332 the Dutch settled here with the permission of the Nawab of Arcot and built a fort called Geldria. For 400 years Pulicat village was buzzing with trade activities with the foreigners and became the envy of the East India Company functioning at Madras. With frequent conflicts, the English finally captured the fort in the year 1795 and destroyed the fort and the Town. What now remain are the remnants of the past. Since this village can be converted as a minor port with the existing light house, and the coastal traffic is contemplated to be increased by deepening the Buckingham Canal which is passing through this village, there is every likelihood that this village will raise as an important Coastal trade centre and it is fitting that this village is given access by an "all weather" road by our Government and the work was inaugurated by Shri M. Bhaktavatsalam, Minister for Agriculture on 11th June, 1955 presided by Shri O. V. Alagesan, then Union Deputy Minister Transport and Railways. The cost of the scheme was 7.01 lakhs.

Description

The length of Ponneri-Pulicat Road is about 11 Miles (3/8 to 15/0) of which 3/8 to 9/5 is an existing metalled road and is a bus route. The reach 9/5 to 12/0 is also an existing road with laterite surfacing but it passes mostly through water logged area due to back water of the sea. The third reach is entirely a new formation and is one vast plain of back water area. During high tide it is completely under water. It is said that there was road across this area and due to Cyclone some 100 years ago, it was obliterated. Any how, no trace of road was visible. The soil is porous with an admixture of soluble salts, silt and fine sand. There are marshy pockets hither and thither. Hence movement of vehicular traffic was difficult and dangerous. So the main bulk of the work was concentrated on the formation of an all weather road for this reach.

There are 20 unbridged water courses interrupting the road at various points. Unless these cross drainages are bridged over, the road formed will not be useful to a maximum extent in all seasons and so proposals were sanctioned for constructing 6 culverts and 14 bridges at an estimated cost of about 5 lakhs. Of these 5 culverts and 5 bridges at a cost of 3 lakhs are under execution and the balance work will be taken subsequently. The formation of the road was

over by March, 1957 despite adverse conditions of weather, especially the cyclonic havoc during 1956-57, unbridged crossings and marshy pockets in the region. In this article it is proposed to give a short "resume" of the works particularly the cross drainage works as executed on that road.

Hydraulic Calculation

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As stated in the previous para, there are about 20 cross drainages on the road apart from other minor culverts and these drainages can be divided into three zones, namely 4/0 to 10/0 (2) 10/0 to 12/0 and (3) 12/0 to 15/1. The first reach consists of cross drainage works among patches of dry and wet fields with outlying storage tanks to irrigate them. The subsoil ranges from sandy loam to dark plastic clay. There are five crossings in this reach at 5/1, 7/1, 8/4, 8/7 and 8/8. These water courses are mostly tank surpluses and act as supply-channels to tanks lower down taking with them the run off from the catchment.

In the second reach, there are six crossings at 10/3, 10/6, 10/8, 11/1, 11/3 and 11/7. Tank surpluses, Spills from low banks of Araniyar River and surface drainage account for the main bulk of the water flow. The soil is chiefly clay of varying consistency and texture.

The third zone lies entirely within the back water area of the sea. Here there is practically no cultivation. The soil is saline with silty sand of a porous structure underlain by organic clay for a considerable depth. There are nine crossings in this reach at 12/4, 12/6, 13/1, 13/3, 13/6, 14/3, 14/5, 14/6 and 15/1. The crossings at 12/4, 14/3 and 15/1 are balancing bridges and four in 13/1, 13/3, 14/5 and 14/6 are only surface drainages, the last two being the run off from the Pulicat village. The most important of these are the crossings across the two arms of the River Araniar at 12/6 and 13/6, branching off prior to consummation with the sea.

Barring Araniar River, the other water courses are simply undefined shallow depressions carrying a wide sheet of water during the monsoon period, but lying fallow during summer. Here and there along the course, moats are formed to impound water for irrigation purposes. The absence of rigid boundaries, defined cross section and uniform bed render it impossible to work out the discharge particulars by the usual method of cross section and velocity. So to design the discharge, the run off from the free catchment as arrived at by the Ryvi's formula and the calculated discharge from tank surpluses wherever they occurred were taken into consideration. The area of the catchment basin was mapped out by tracing the several water courses marked in the ordnance Map and assuming them to be the ridge lines of the area. This procedure is adopted due to the absence of topographical features in the map as the region is very close to the sea. The design discharge is then found out by the use of Ryvi's formula. Having thus found out, the next step was to determine the linear water way and its depth by using "Lacey's theory of Flow in Incoherent Alluvium", a brief account of which will not be out side the scope of this article.

Lacey's theory maintains that alluvial streams that meander a good deal over a wide belt get regimented in course of time and acquire, by repeated process of scouring and silting, a stability in such a cross section and such a slope that the resulting velocity will be non-silting and non-scouring. When such a stability occurs, the water course is said to have come to regime. In other words, an alluvial stream carrying a known discharge Q would have a regime wetted perimeter " P " or a regime width " W ", a regime slope " S " and a regime hydraulic depth " R " or scour depth " D ", with the result that the area of cross section " A " and the velocity " V " will become fixed. In such a situation, the silt factor " f " plays a decisive role in the relationship between P , R , S , V and A . Following are some of the relationships:

$$P = 8/3 Q^{\frac{1}{3}} \quad \text{or} \quad W = 8/3 Q^{\frac{1}{3}} \quad \dots (1)$$

$$R = \frac{473 Q^{\frac{1}{3}}}{f^{\frac{1}{3}}} \quad \text{or} \quad D = \frac{0.473 Q^{\frac{1}{3}}}{f^{\frac{1}{3}}} \quad \dots (2)$$

$$S = \frac{0.00542 f^{\frac{5}{3}}}{Q^{\frac{1}{3}}} \quad \dots \quad \dots (3)$$

$$V = 0.794 Q^{\frac{1}{3}} f^{\frac{1}{3}} \quad \dots \quad \dots (4)$$

$$A = \frac{1.26 Q^{\frac{2}{3}}}{f^{\frac{1}{3}}} \quad \dots \quad \dots (5)$$

Normally for alluvial streams the regime width is adopted as the lineal water way. But if circumstances do permit the linear water way " L " is kept less than the regime width " W ", when the normal scour depth " D " will change over to D_1 " D_1 ", as given below:

$$D_1 = D \left(\frac{W}{L} \right)^{0.61} \quad \dots \quad \dots (6)$$

The following are the rules that are usually followed in fixing the maximum scour depth.

1. One and half times the normal scour depth modified for the effect of contraction where necessary—When the course is straight with provision for a single span.

2. Twice the normal scour depth modified for the effect of contraction where necessary, in case of bad sites with diagonal currents and multi-span structure.

3. In cases of bridges where " L " is less than " W ", the maximum scour depth obtained by rules 1 and 2 should be compared with the following formula and the greater of the two values taken:

$$D_m = D \left(\frac{W}{L} \right)^{1.56} \quad \dots \quad \dots (7)$$

where

D_m = Maximum scour depth

D = Normal scour depth

L = Linear water way

W = Regime width.

In fixing the depth of foundations the following are the guiding principles:

Erodible soils

$1\frac{1}{2}$ depth of the maximum scour.

Hard Beds

$1' 0"$ into rock or $2' 0"$ into other hard stratum if it is met with at a higher level than the limits given for erodible soils;

Note: In both the cases, the soil at which the structure is founded should have sufficient bearing power.

With these principles, the design of the bridges has been made and the design is described in subsequent paras.

Design of Foundations

Undisturbed soil samples of the foundation soil were taken by the staff of the Highway Research Station and suitable recommendations were made by the Director, on the test findings. On the close perusal of the recommendations (Annexure A), the following facts are clearly borne out.

(a) If the natural moisture content of the foundation soil is nearer to Plastic limit than liquid limit it can be taken as a favourable condition to go in for open foundation with footings, whose bottom width should, of course, be spread out to commensurate with the bearing power of soil.

(b) If the natural moisture content of the foundation soil is nearer to liquid limit than to plastic limit then it is an unfavourable condition, since the soil will be soft with low bearing power. In such cases provision of block concrete would distribute the load on foundation over a wide area so as to bring down the stress on soil well within its bearing power.

(c) When the natural moisture content and the liquid limit are almost equal, then it denotes the presence of excessive colloidal clay in the composition of the soil and it is a sure indication of poor bearing power. Under such conditions, there are three courses open to provide a stable foundation:

1. To lay block concrete over a sand cushioning of adequate depth to disperse the load over an area which will be sufficient to keep the stress within safe bearing capacity of the soil.

2. To go in for well foundations and

breaking of pile head. Then the hammer is operated to deliver blows that drive the pile into the ground.

Observations have been made and recorded of the behaviour of sinking of piles through different stratum of soil. These observations can be seen as Annexure D. The bearing power of a pile is interpreted from the average set per blow for the last 10 blows. There are several empirical formulas in vogue to deduce the safe bearing power of a pile depending on its weight, weight and drop of hammer and the average set per blow. The resistance of a pile due to soil friction against penetration is not the only criteria for resting a pile. A pile has to go down sufficiently deep into the ground well beyond the range of scour depth, so that the surrounding soil will develop the requisite frictional resistance. Also if the set for the first few blows when driving is recommended after a few days differs from the set at the end of the first driving, the ground is likely to alter its resistance with time and so, much reliance cannot be placed on a dynamic formula. So the results obtained from a formula can only be taken as a rough guide and the ultimate bearing power should however be vouchsafed only upon conducting a load test. This argument can be verified by taking the use of two such formulas on the data from bridge at 12/6.

Data

Test pile, length, size and weight 20 ft. 12"×12" and 3000 lbs. Weight of gravity hammer used 2240 lbs. drop of hammer 3' 0".

Average set for the last 10 blows : 1/6".

Total No. of blow delivered to drive the pile : 404.

1. According to the formula given in S. S. R. B. (1005-13).

$$P = \frac{2WH}{S \left(1 + \frac{W_p}{W}\right)}$$

Where P Safe bearing power in pounds
W Weight of hammer in pounds
H drop in feet
W_p weight of pile.
S Average set for the last 10 blows in inches.

$$= \frac{2 \times 2240 \times 3}{1/6 \left(1 + \frac{3000}{2240}\right)} = 15.3 \text{ Tons.}$$

2. By the simplex formula

$$U = \frac{N}{L} \times \frac{HW}{1+P} \times \sqrt{\frac{L}{50}}$$

U Ultimate load
N Total No. of blows to drive the pile
L length of pile in feet
H Drop in feet
P penetration for last blow
W Weight of hammer in pounds.

$$= \frac{404}{20} \times \frac{3 \times 240}{1 + \frac{1}{6}} \times \sqrt{\frac{20}{50}} = 32.8 \text{ tons.}$$

Since the soil through the pile was driven is soft, a factor of safety of 4 is adopted and hence the safe bearing power will be 8.2 tons.

The variation between the two results is appreciable. No definite equation has yet been evolved to put into exact mathematical terms the relationship between the sinking of the pile under a static load and a kinetic hammer load. This will be further evident from the results of the tests conducted in the bridge works in 12/6 and 13/6.

The design load on each pile is 10 tons. According to S. S. R. B. 1005-13 the test load should be twice the design load and after 48 Hours of Static application of this load, there shall not be a settlement more than 1/4".

In the bridge work at 12/6 the actual load placed over the test pile was 22 tons applied with a specially fabricated platform and after 48 hours there was no settlement at all. According to the formula as worked out earlier for the same pile, the bearing capacity is only 8.2 tons which proves the statement made earlier.

Again in 13/6 work, the test load was conducted in two stages on the same pile. The length of the test pile was 20 feet. The first stage consisted of driving the test pile to the required elevation of 67.50 when the average set was 1/2". Accordingly the empirical formula No. 1 gave a safe bearing capacity of 7 tons. Making allowance for the extra frictional resistance due to the length between the top of the pile at the designated level and the top of the loading platform (i.e. the height which will be uncovered while laying capping slab) a test load of 27.2 tons was applied. After 48 hours a settlement of 1.8" was observed which was far in excess of allowable limit. So the pile was driven to 4 feet lower, when the average set was 0.2" per blow. As per formula 1, the safe load worked out to be 17 Tons. The test load of 27.2 tons was applied and no settlement was observed after 48 hours.

The behaviour of the test piles in different soils as recorded in these bridge works can be clearly seen from the observations enclosed.

Conclusions

In this short article, I have endeavoured to present a glimpse of the improvements to Ponneri-Pulicat Road, some of which are still in progress. Of course, there are many works progressing in our State bigger in magnitude and modern in techniques but one special feature of the works on this road is the difficult terrain and marshy grounds on which the foundation problem has been successfully tackled. It is my sincere conviction that in such cases, where open foundations cannot be thought of, and well foundations will be highly expensive, adoption of pile foundation is the only economical way especially for light structures.

My grateful thanks are due to the Director of Highways Research Station, Guindy and the then Divisional Engineer (Highways) Saidapet for inducing me to bring forth this article and for their valuable suggestions to finalise it.

3. To have pile foundation with precast R. C. C. piles of sufficient length and number and a capping slab over them for even distribution of the load.

These aspects are dealt separately in a subsequent para.

(d) Where a structure is found on sand it is best to have block concrete to withstand the adverse effects of piping action.

(e) Where a foundation soil contains a good proportion of granular material like sand the bearing power will be good because of the internal friction it develops. But if the soil contains more of plastic material like clay, the internal friction will be very small and negligible and consequently its bearing power will also be low.

Since the foundation soil met with in most of the cases has got natural moisture content nearer to Liquid limit, suitability of different foundations (as detailed in para c-2 above) was investigated. It was found that adoption of pile foundations will be most economical considering the quickness in execution, cost and availability of the equipments and the same type of foundation was adopted uniformly thereby considerable economy has been effected.

Choice of Spans

Due to poor condition of foundation soils, many proposals were worked out from installation of Hume pipe of various diameter to R. C. C. slab structure and box culverts. Finally 10' 0" spans of R.C.C. slab were adopted on an uniform basis which proved very economical. Since these water courses carry no floating materials, the free board of the span was not given as required in the standards but was restricted to the extent that it would not create a hog-back in the longitudinal profile of the Road.

A detailed drawing of the bridge in Mile 11/7 is given as Annexure (B) which is representative of the typical design adopted for bridges on this Road.

Design and Construction of Piles

Generally there are two types of piles, bearing piles and friction piles. The function of a pile is to transmit the load of the structure to the subsoil. It can behave either as a friction pile or end bearing pile, depending mainly on the carrying capacity due to surface friction or its action as a column resting on a firm bed. Friction piles which do not penetrate down to a firm strata are not very effective unless they are at least as long as the width of the structure that they support. A precast pile has to be designed to satisfy the following conditions. It shall be strong enough to resist the stresses caused by (a) the ultimate load acting on the pile (b) the bending stresses during handling and lifting due to its self-weight and (c) the dynamic stresses produced during pile driving.

A typical design of a precast pile as used in these works is given in Annexure C.

Pile Driving

In the cross drainage works on this road, two different methods of pile driving were adopted. One was of simple type with a drop hammer and the other is a B. S. P. make with No. 2 Diesel hammer owned by the Department. Of the two type judged from all points of working facility, the plant of the latter type proved to be the best one and a brief description of its assembly and its working will be of some interest to those who will be using such kind of Plant.

The B.S.P. Make Plant

A general view of the plant can be seen from the skeleton drawing. It is a light type designed for economy in operation and in transport and possesses the special feature of being self erecting with the aid of its own winch.

The overall height is 35' 0" with a base of 14' 0" × 9' 0". The frame consists of a pair of channels forming the leaders connected to another pair, which together with the raking struts forms the back strut. The leaders are guyed sideways to the base and the frames are mounted on rollers. The total weight is 72 cwt. excluding weight of hammer and engine. The ballast required to keep the plant balanced and steady while driving is 3 tons consisting of sand bags. Trackways for the plant to move on rollers were provided with longitudinal timber sleepers 0' 8" × 0' 8" and in length of 8' 0" to 10' 0" each piece connected together with an overlap of 1' 6".

The Hammer

Pile driving is done by an engine driven winch. Of the three types of hammers in general use, namely (1) Drop (gravity) hammer (2) Single acting hammer and (3) the double acting hammer, the B. S. P. No. 2 Diesel hammer is single acting but it can be used as a drop hammer also. The hammer is started by raising the ram to the top of its stroke on the triphook and tripping it to fall freely. In falling to deliver its blow, the ram also compresses the air trapped above the piston and the injected charge of the diesel oil to the point of ignition. The ram is returned to the top of its stroke by the expansion of its products of combustion and falls to deliver another blow and repeat the cycle of operations continuously. This repeated action is found to be very effective when the soil friction resists the penetration of the pile. When the automatic action is on, the hammer can easily be stopped by simply pulling the reset sliding block out of contact with the fuel pump tappet rod. An anvil block is coupled with the hammer. It is a self contained unit and needs no external source of power other than the winch to lift the hammer as a whole.

Operation

The piles to be driven are usually kept as near the plant as possible, so that they will be within easy reach of the launching wire rope. After the pile is hoisted and placed in position to the plumb, the anvil plate of the Hammer is lowered down to the pile head to press it. A cushioning pad of jute cloth 3" thick is placed on the pile head before placing the anvil to prevent edge

**Annexure A—Puduvayal Pulicat Road-Cross Drainage Works-Tabular Statement
Showing Laboratory findings and recommendations**

Serial No.	Location of Dip.	Still level as proposed by D. R. (H) Saidapet.	Foundation level as proposed by the D. R. (H) Saidapet.	Nature of foundation soil.	Discussion of soil profile test results and bearing capacity.	Recommendations.	Final proposals approved by the Chief Engineer(Highways) and adopted for execution.
1	Dip at 5/1 of the Road	94.00	88.00	Stiff dark brown clay	The soil profile consists of 1½' thick of coarse sand at the top underlain by 4½' thick of brown sandy clay and more than 9½' thick of stiff dark down clay. The brown sandy clay has a natural moisture content of 26% which is nearer to the plastic limit (15%) than the liquid limit (40%). The bearing capacity of the soil is 0.35 T/sq. ft. The stiff dark brown clay on which the proposed foundation is laid is a stiff dark brown clay with a Natural moisture content (17%) very near to the plastic limit (16%). This is a favourable condition and the bearing power of the soil is also 2.1 T/sq. ft.	The maximum stress coming on the soil is only 0.98 T/sq. ft. as against the safe bearing capacity of 2.1 T/sq. ft. of the soil. Hence the proposal of the Divl. Engr. (H), Saidapet to have the foundation at 88.00 on stiff dark brown clay may be adopted.	5 spans of 10' 0" each Open foundations with footings
2	Dip at 7/6-7 of the Road	93.50	89.00	Medium stiff/ brown sandy clay	The soil profile consists of about 1' thick of brown sandy clay underlain by more than 14' thick of brown sandy clay.	The intensity and pressure is 0.4 T/sft. against safe bearing capacity of 0.79 T/sft. of the soil.	4 spans of 10' 0" each

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IMPROVEMENTS TO PONNERI - PULICAT ROAD

3	Dip at 8/4 of the Road	97.50	93.00	Coarse sand	The soil at the foundation elevation is medium stiff brown sandy clay whose natural moisture content (33%) is nearer to plastic limit (24%) than liquid limit (49%). This is a favourable condition. The bearing power of the soil is 0.79 T/sq. ft. The top soil in this dip is 2' thick of brown sandy clay underlain by 3' thick of brown clay and more than 10' thick of coarse sand. The brown clay has got a natural moisture content of 40% which is nearer to the liquid limit (54%) than plastic limit (17%). The bearing power is also poor (0.43 T/sq. ft.). The foundation soil is dense sand with a bearing capacity of 3.5 T/sq. ft.	Block concrete at 89.00 as proposed by the Divisional Engineer (H) Saidapet may be adopted. The proposal of the Divl. Engineer (H) Saidapet to have the foundation at 93.00 on coarse sand which has a safe bearing capacity of 3.5 T/sq. ft. against the stress of 0.98 T/sq. ft. may be adopted. Comparative cost with box culvert with sill at 98.00 may also be worked out and the cheaper of the two proposals can be adopted.	Open foundations with block concrete 3 spans of 10' 0" each Open foundations with footings
4	Dip at 8/7 of the Road	93.00	89.00	Dark brown clay	The soil profile consists of 2' thick of brown clay with 2' thick of dark grey sandy clay and not less than 8' thick of brown clay underneath. The dark grey clay is soft with its Natural moisture content close to the liquid limit. It has got poor bearing capacity value of 0.24 T/sq. ft.	The proposals of the Divl. Engineer (H) Saidapet to have the foundation with independent footings at Ele. 89.00 on dark brown clay having a safe bearing capacity of 1.39 T/sq. ft. against a stress of 0.9 T/sq. ft. may be adopted.	2 spans of 10' 0" each Open foundations with footings

**Annexure A—Puduvayal Pulicat Road-Cross Drainage Works-Tabular Statement
Showing Laboratory findings and recommendations—(Contd.)**

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Serial No.	Location of Dip.	Sill level as proposed by D. M. (H) Saidapet.	Foundation level as proposed by the D. M. (H) Saidapet.	Nature of foundation soil.	Discussion of soil profile test results and bearing capacity.	Recommendations.	Final proposals approved by the Chief Engineer(Highways) and adopted for execution.
5	Dip at 8/8 of the Road	91.70	84.00	Sand with small percentage of silt and clay	The soil at the foundation elevation is dark brown clay with Natural moisture content very near plastic limit. The bearing power of the soil is also 1.39 T/sq. ft. The soil profile for this bore consists of 5' thick coarser sand underlain by 2' thick of soft dark grey clay and not less than 13' thick of sand with small percentage of silt and clay. The dark grey clay is soft with its natural moisture content (44%) nearer to liquid limit (57%) than plastic limit (22%). The bearing power is also poor (0.24 T/sft.). The soil at the foundation elevation is sand with a small percentage of silt (10%) and clay (1%) the bearing power at the soil is 3.5 T/sft.	The foundation proposed is on sand with a small percentage of silt and clay having safe bearing capacity of 3.5 T/sft. The bottom of foundation proposed is at 84.00. Instead of this the foundation can be laid at Ele. 88.00.	5 spans of 10' 0" each Open foundations with footings

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6	Dip at 10/3 of the Road	96.00	92.00	Sand with smaller percentage of silt and clay	The top soil in this dip is 3' thick of brown clay and underneath it sand with small percentage of silt and clay is available for a height of about 17'. The brown clay has got a Natural moisture content nearer to the liquid limit and is soft. The bearing capacity of the soil is 0.43 T/sq. ft. The foundation soil is sand with a small percentage of silt (10%) and clay (1%). The bearing capacity of the soil is 3.5 T/sft.	The proposal of the Divisional Engineer (H) Saidapet to have the foundation on sand at Ele. 92.00 may be adopted. The safe bearing capacity of the soil is 3.5 T/sq. ft. against stress of 0.98 T/sft. on the soil.	2 spans of 10' 0" each Open foundations with footings
7	Dip at 10/6 of the Road	96.00	91.00	Sand with small percentage of silt and clay	The soil profile consists of 5' thick of brown clay and about 15' thick of sand with silt and clay. The brown clay is very soft with its Natural moisture content close to the liquid limit. The bearing capacity of the soil is 0.43 T/sft. The foundation soil is sand, dark grey in colour and with a small percentage of silt (10%) and clay (1%). The bearing power of the soil is 3.5 T/sft.	The safe bearing capacity of the sand with small percentage of silt and clay is 3.5 T/sft. against stress of 0.98 T/sft. coming on it. The proposal of the Divl. Engineer (H) Saidapet to lay the foundation on the sand at Ele. 91.00 may be adopted.	2 spans of 10' 0" each Open foundations with footings

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**Annexure A—Puduvayal Pulicat Road-Cross Drainage Works-Tabular Statement
Showing Laboratory findings and recommendations—(Contd.)**

Serial No.	Location of Dip.	Still level as proposed by the D.E. (H) Saidapet	Foundation level as proposed by the D.E. (H) Saidapet	Nature of foundation soil.	Discussion of soil profile test results and bearing capacity.	Recommendations.	Final proposals approved by the Chief Engineer(Highways) and adopted for execution.
8	Dip at 10/8 of the Road	96.50	91.00	Sand with small percentage of silt and clay	The soil profile consists of 5' thick of brown sandy clay underlain by about 10' brick of sand with silt and clay. The natural moisture content of the brown sandy clay is nearer to the plastic limit but the bearing power of the soil is low (0.35 T/sq. ft.) The foundation soil which is sand, dark grey in colour and with a small percentage of silt and clay has got a bearing power of 3.5 T/sft.	The proposals of the Divl. Engineer (H) Saidapet to have the foundation on sand with a small percentage of silt and clay at Ele. 91.00 may be adopted.	1 span of 10' 0" Open foundations with block concrete
9	Dip at 11/1 of the Road	95.00	90.00	Dark grey organic clay	In this dip the soil profile is made up of 6' thick of brown clay underlain by 4' thick of dark grey organic clay and not less than 10' thick of sand. The brown clay has got a natural moisture content (35%) which	The proposals of the Divisional Engineer (H), Saidapet to provide 5' sand and with 2' block concrete may be adopted.	3 spans of 10' 0" each Open foundations with sand filling and block concrete

10	Dip at 11/7 of the Road	93.00	89.00	Dark grey sandy clay	is nearer to the liquid limit (48%) than the plastic limit (16%). The bearing power of the soil is 0.8 T/sq. ft. The soil at the foundation elevation is dark grey clay containing 8% organic matter. The natural moisture content of the soil (77%) is very high and very near liquid limit (87%). The clay is highly plastic and the specific gravity is 2.3. The bearing capacity of the soil is sand with small percentage of silt (10%) and clay (1%). The bearing power is 3.5 T/sq. ft. The top soil in this dip is 3' thick of brown clay and more than 14' thick dark grey sandy clay. The brown clay has got a natural moisture content very close to the liquid limit and is very soft. The bearing is also poor (0.43 T/sft.). The soil at the foundation elevation is dark grey sandy clay whose natural moisture content (20%) is very near liquid limit (22%). Also it has got a poor bearing power (0.21 T/sft.).	From Ele. 89.00 to 83.00 of the existing soil may be excavated and filled with sand with additional dimensions for dispersion at 45°. Depending upon the case of execution block concrete may be adopted. Box culvert may work out comparatively cheaper. A second alternative is to have concrete piles 6" x 6" carried down to 12'.	4 spans of 10' 0" each Precast R.C.C. pile foundation—R.C.C. piles 12" x 12" 15' 6" long spaced at 3-9' centres
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**Annexure A—Puduvayal Pulicat Road-Cross Drainage Works-Tabular Statement
Showing Laboratory findings and recommendations—(Contd.)**

Serial No.	Location of Dip.	8111 level as proposed by D. E. (B) Saldaput.	Foundation level as proposed by the D. E. (B) Saldaput.	Nature of foundation soil.	Discussion of soil profile test results and bearing capacity.	Recommendations.	Final proposals approved by the Chief Engineer(Highways) and adopted for execution.
11	Dip at 11/7 of the Road	90.00	86.00	Dark grey sandy clay	The top soil in the dip is medium sand $1\frac{1}{2}$' in height. Beneath this brown sandy clay of $5\frac{1}{2}$' thick, dark grey clay of $10'$ thick and reddish brown sandy clay at more than $8'$ thick are available. The brown sandy clay is of soft nature with its natural moisture content (20%) near liquid limit (23%). The bearing capacity of the soil is 0.32 T/sq. ft. The dark grey sandy clay is also of soft variety with its natural moisture content (20%) very near liquid limit (22%). The bearing power of the soil is 0.34 T/sq. ft. In the case of reddish brown sandy clay the natural moisture content (16%) is closer to the liquid limit (20%) than plastic limit (9%). The bearing power of the soil is 0.8 T/sq. ft.	R. C. Piles $6" \times 6" \times 10'$ to $12'$ long, spaced at $2'$ clear either way with top $2'$ sand filling and block concrete $2'$ thick, with the pile heads carried 6 inches into the concrete block may be adopted.	9 spans of $10' 0"$ each Precast R. C. pile foundations. R. C. piles $12" \times 12"$ and $15' 6"$ long Spaced at $3' 9"$ centres

12	Dip at 12/4 of the Road	89.00	85.00	Dark grey sandy clay	The soil profile in this bore consists of $\frac{1}{4}$ thick of brown sandy clay underlain by $1\frac{1}{2}'$ thick of dark sandy clay and $4\frac{1}{2}'$ thick of reddish brown sandy clay. The dark grey sandy clay has a natural moisture content (20%) near to liquid limit (22%) the bearing capacity of the soil is very low (0.21 T/sq. ft.). The reddish brown sandy clay has got natural moisture content close to the liquid limit. The bearing capacity of the soil is 0.8 T/sq. ft.	Similar proposed as for item 11 may be adopted with the top of bed blocks $2'$ below sill i.e. at 87.00.	2 spans of $10' 0"$ each Precast Pile foundations R. C. piles $12" \times 12"$ and $15' 6"$ long spaced at $3' 9"$ centres
13	Dip at 12/6 of the Road	86.00	82.00	Dark grey sandy clay	The soil profile consists of $4'$ thick of fine sand underlain by $13'$ thick of dark grey sandy clay and more than $10'$ thick of reddish brown sandy clay. The natural moisture content of the dark grey sandy clay (37%) is nearer to liquid limit (40%) and its bearing power is 0.72 T/sq. ft. The reddish brown sandy clay has got a natural moisture content (16%) which is nearer to plastic limit (10%). The bearing power of the soil is 0.72 T/sq. ft.	R. C. Piles and concrete bed block $2'$ thick with top level at 84.00 may be adopted. The piles may have to be taken down into red brown clay and tested for safe bearing capacity comparative costs may be worked out against box culvert.	10 spans of $10' 0"$ each Pile foundations R. C. piles $12" \times 12"$ and $16' 6"$ long Spaced at $3' 9"$ centres

**Annexure A—Puduvayal Pulicat Road-Cross Drainage Works-Tabular Statement
Showing Laboratory findings and recommendations—(Contd.)**

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Serial No.	Location of Dip.	Sill level as proposed by D. M. (H)	Foundation level as proposed by D. M. (H)	Nature of foundation soil.	Discussion of soil profile test results and bearing capacity.	Recommendations.	Final proposals approved by the Chief Engineer(Highways) and adopted for execution.
14	Dip at 13/1 of the Road	90.00	86.00	Dark grey sandy clay	The soil profile consists of 3 1/2' thick of brown sandy clay and more than 16' thick of dark grey sandy clay. The dark grey sandy clay is of soft variety with its natural moisture content (20%) closer to the liquid limit (22%). The bearing power of the soil is 0.21 T/sq. ft.	The proposals of the Divisional Engineer (H) Saidapet to have the block concrete foundation at 86.00 after excavating the soft soil and filling with sand may be adopted.	1 span of 10' 0" Precast pile foundation R. C. piles 12" x 12" and 15' 6" long Spaced at 3' 9" centres
15	Dip at 13/3 of the Road	89.00	85.00	Dark grey sandy clay	The soil profile consists of 2' thick of brown clay underlain by 2' thick of medium sand and more than 12' thick of dark grey sand clay. The natural moisture content of the dark grey clay (20%) is nearer to the liquid limit (22%). The bearing power of the soil is 0.21 T/sq. ft.	The proposals of the Divisional Engineer (H) Saidapet to have the block concrete foundations at 85.00 after excavating the soft soil and filling with sand may be adopted.	1 span of 10' 0" Precast pile foundation R. C. piles 12" x 12" and 15' 6" long spaced at 3' 9" intervals.

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16	Dip at 13/6 of the Road	86.00	82.00	Dark Grey clay	The soil profile consists of 4' thick of fine sand underlain by 1' thick of soft brown clay, 15' thick of dark grey clay, 2' thick of sand with a small percentage of silt and more than 4' thick of Reddish brown sandy clay. The natural moisture content (66%) of the dark grey clay is nearer to the liquid limit (69%) and is soft. The bearing power of the soil is 0.5 T/sq. ft. The dark grey sand containing a small percentage of salt (5%) has got bearing capacity of 3.5 T/sq. ft. The reddish brown sandy clay has got a natural moisture content of (16%). The bearing capacity is 1.34 T/sq. ft.	In the proposals made by the Divisional Engineer (H) Saidapet 6 vents of 10' have been provided and the foundation is to be of block concrete at Ele. 82.00 after excavating the clay and filling with and to a depth of 6'. The natural moisture content of the soil is very high and very near liquid limit. It may be difficult to excavate and fill up with sand in such a type of soil. Either a multiple box culvert or pile foundation may be adopted.	6 spans of 10' 0" each Precast pile foundation R. C. piles 12" x 12" and 19' 6" long spaced at 3' 9" intervals
17	Dip at 14/33 of the Road	87.00	83.00	Sand with small percentage of silt	The top soil consists of medium sand (4 1/2' underlain 16' thick of sand with a small percentage of silt and 4' thick of brown sandy clay). The foundation soil is sand dark grey in colour and with small percentage of silt (5%). It has got a bearing capacity of 3.5 T/sq. ft.	The proposals of the Divisional Engineer (H) Saidapet to have the foundation with independent footings at 83.00 on sand with a small percentage of silt and having a safe bearing capacity of 3.5 T/sq. ft. may be adopted.	3 spans of 10' 0" each Open foundations with footings

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**Annexure A—Puduvayal Pulicat Road-Cross Drainage Works-Tabular Statement
Showing Laboratory findings and recommendations—(Contd.)**

Serial No.	Location of Dip.	Fill level as proposed by D. M. (H) Saldapet.	Foundation level as proposed by the D. M. (H) Saldapet.	Nature of foundation soil.	Discussion of soil profile test results and bearing capacity.	Recommendations.	Final proposals approved by the Chief Engineer(Highways) and adopted for execution.
					The brown sandy clay has got a natural moisture content of 18% which is near to the liquid limit (22%) The bearing capacity of the soil is 1.4 T/sq. ft.		
18	Dip at 14/5 of the Road	90.00	86.00	Fine sand	The soil profile consists of 16' thick of fine sand and more than 8' thick of sand. The fine sand is medium dense and its bearing capacity is 1.26 T/sq. ft. The sand underneath this is dark grey in colour and contains a small percentage of silt and clay. The bearing capacity of the soil is 3.5 T/sq. ft.	It is proposed by the Divl. Engineer (H) Saldapet to lay the foundation at 86.00 on fine sand. Though the safe bearing capacity of the soil is good viz. 1.26 T/sq. ft. scour considerations must be taken into account. The proposals may be adopted after making sure about the probable scour. If necessary the foundations may be taken down.	1 span of 10' 10" Open foundations with footings

19	Dip at 14/6 of the Road	90.00	86.00	Sand with a small percentage of silt	The soil profile consists of 6' thick of fine sand and 14' thick of sand with a small percentage of silt The foundation soil is sand containing a small percentage of silt (2%). The bearing capacity of the soil is 3.5 T/sq. ft.	Similar proposals as in item 18 above may be adopted.	1 span of 10' 0" Open foundations with footings
20	Dip at 15/1 of the Road	85.00	80.00	Sand with a small percentage of silt and clay	The soil profile consists of 2' thick of brown sandy clay underlain by 2' thick of medium sand 9' thick of sand with silt and clay and more than 7' thick of coarse sand with shells. The foundation soil is sand grey in colour and containing silt (12%) and clay (7%). The bearing power of the soil is 1.3 T/sq. ft.	The proposals of the Divl. Engineer (H) Saldapet to lay the foundation at 80.00 on sand with a small percentage of silt and clay having safe bearing capacity of 1.37 T/sq. ft. may be adopted.	Dropped

ANNEXURE C.

Name of work: Construction a bridge Across Araniar @ mile 12/6 of Pudukkottai Road.

C. S. at the combined stream at L. S. 1250 of the stream.

OFL	RL	Diff.	Sum of successive differences	Mean differences	Length	Area
92.58	91.00	1.58	1.58	0.79	3	2.37
	89.18	3.40	4.98	2.49	7	17.43
	86.18	6.00	9.40	4.70	20	94.00
	86.72	5.86	11.86	5.93	10	59.30
	86.88	5.76	11.82	5.91	10	59.10
	87.08	5.50	11.26	5.63	10	55.30
	87.10	5.48	11.98	5.49	10	54.90
	87.40	5.18	10.66	5.33	10	53.30
	87.58	5.00	10.18	5.09	10	50.90
	86.66	5.92	9.92	4.96	10	49.60
	88.88	3.70	8.62	4.31	20	86.20
	89.00	3.58	7.28	3.64	10	36.40
	89.88	2.70	6.36	3.18	10	31.80
	91.90	0.68	3.46	1.73	4	6.92
					144	658.52

$$H. M. D. R = A/P = 658.52/134.00 =$$

$$R = 4.914 \quad R^{\frac{1}{4}} = 2.217 \quad n/(r)^{\frac{1}{4}} = 0.025/2.217 = 0.01128$$

$$\text{Using Kutter's formula } C = \frac{41.66 + 0.00281/S + 1.811/n}{1(41.66 + 0.00281/S) n/r^{\frac{1}{4}}}$$

$$\text{Bed slope } S = 92.50 - 92.00/1250 = 0.5/1250 = 1/2500 = 0.0004$$

$$\text{By Manning's formula } n = 0.025; r = 4.914; S = 0.0004$$

$$V = 1.486/n \times r^{2/3} S^{\frac{1}{2}} = 3.55 \text{ ft./sec.}$$

$$\text{Discharge: } 659 \times 3.55 = 2340 \text{ cusecs or } 2400 \text{ cusecs.}$$

ii. Design of Ventway

$$H. F. L. 92.00; \text{ Sill level: } 86.00; \text{ Height of flow } 6.00$$

$$\text{Regime width by Lacy's formula } = 5/3 G^{\frac{1}{2}} 2.67 \times 2400^{\frac{1}{2}} = 131 \text{ feet.}$$

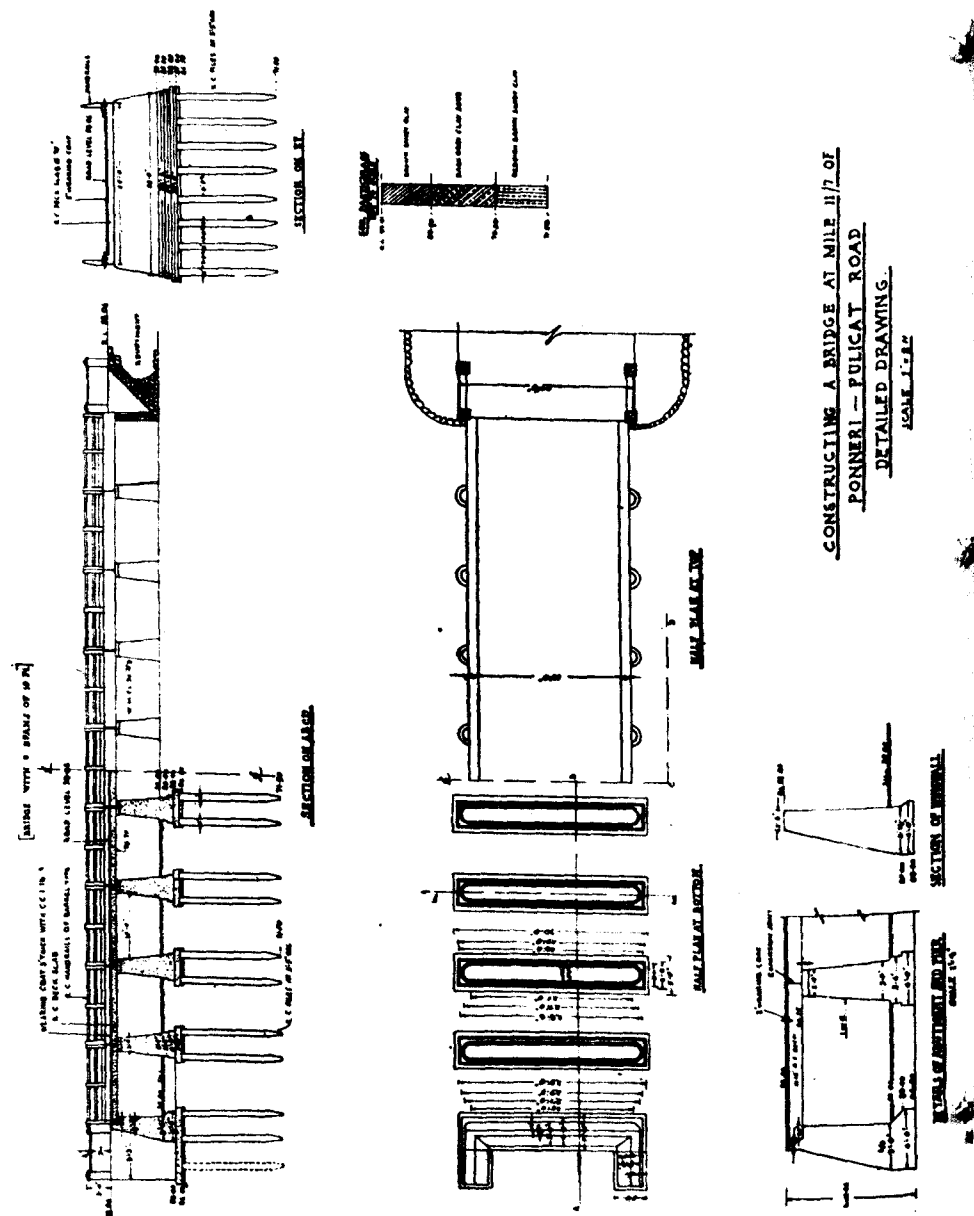
Hence 10 spans of 10 feet clear each are adopted.

$$\text{Velocity developed in the vents } 2400/9 \times 10 \times 10 \times 6 = 4.44 \text{ ft./sec.}$$

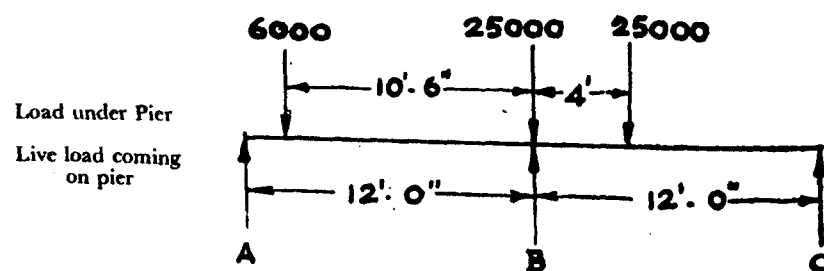
This velocity is allowable.

iii. Check for scour

Since the bed is clayey which is non erodible the structure is safe against scour.



Pile foundation for the Bridge at 12/6 of Ponneri Pulicat Road



Pressure on B: $25000 + 25000 \times 8/12 + 6000 \times 1.5/12$

$$= 25000 + 16667 + 750 = 42417 \text{ lbs.} = 18.94 \text{ say } 19 \text{ tons.}$$

For 2 lane roadway: $2 \times 19 = 38 \text{ tons.}$

Dead load on pier:

Deck slab: $12 \times 251 \times 1.06 \times 150/2240$	...	23.31
Wearing coat: $12 \times 22.0 \times 0.25 \times 14.0/2240$	...	4.13
Kerb: $2 \times 12 \times 1.5 \times 1 \times 150/2240$	...	2.41
Hand rails: $2 \times 12 \times 100/2240$	...	1.07
		30.92
	say ...	31 Tons.

Weight of pier;

I footing: $29.50 \times 4.5 \times 1 \times 140/2240$	...	8.296
II footing: $29.00 \times 4.0 \times 1 \times 140/2240$	...	8.156
Above sill $25.00 \times (3.6 + 2.0) \times 12 \times 7 \times 140/2240$	...	30.07
Capping slab: $30.00 \times 5.75 \times 1.5 \times 150/2240$	...	17.25
		63.772
	say ...	64 tons.

Total downward weight: $38 + 31 + 64 = 133 \text{ Tons.}$

Horizontal load:

(a) Horizontal force due to water currents: $P = KAV^2$

Where $K = 0.66$ $A = \text{exposed area}$; $V = \text{mean velocity}$

$$A = 8 \times (3' 6" + 2' 6")/2 = 24 \quad V = 3.55 \text{ ft./sec.}$$

Therefore Thrust: $0.66 \times 24 \times 3.55^2 = 135 \text{ lbs.}$

(b) Horizontal forces due to wind load:

$P = AP$: A : exposed area P : Pressure due to wind

$$P. (12 \times 24/2 + 12 \times 12/12) \times 20 = 720 \text{ lbs.}$$

Since the magnitude of the horizontal loads is very small compared to that of the vertical loads and that the moment of inertia of the pier in the corresponding direction is great the effect due to horizontal loads on soil pressure on foundation is negligible.

III. Longitudinal force on pier is $5 + 0.12 \times 12 = 6.44 \text{ Tons.}$

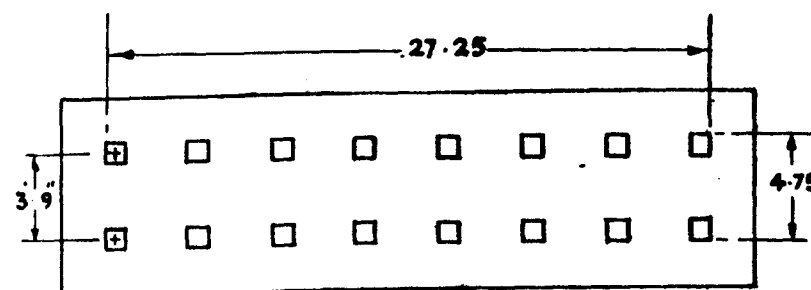
B. M. due to longitudinal force: $6.44 \times 11.7 = 75.34 \text{ say } 71 \text{ ft. tons.}$

Load on each pile: $W/n + M/I$ where W is total vertical load;

n : : No. of piers.

M : Bending moment I : : moment of inertia of pile group.

Assuming a pile group of this shape.



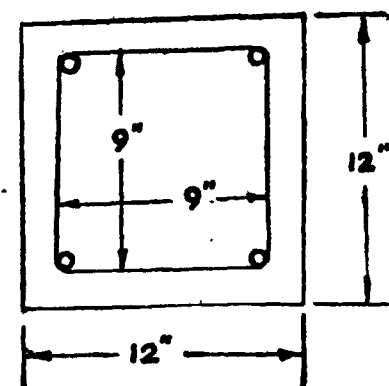
$$I: : 16 \times bd^3/12 + 16 \times 3.75^2/2^2$$

$$= 16 \times 1/12 + 56.25: 57.58 \text{ pile ft.}^3$$

Load on each pile: : $133/16 + 71/57.58 = 8.31 + 1.23 = 9.54 = \text{say } 10 \text{ tons.}$

Which is piles much less than the maximum allowable load on concrete of 40 tons.

The arrangement of piles are effective since the bounding perimeter of the pile group is 64 ft. and the sum of perimeters of piles 64 feet, taking the size of pile as $12" \times 12"$.



Assuming that the length of piles is 15 feet, the piles remain in dark grey sand of clay for 8 feet and reddish brown sandy clay for 7 feet.

(a) The bearing power of dark grey sandy clay is 0.72 tons unconfined compressive stress: $0.72 \times 1.5 = 1.08$ tons.

Ultimate skin friction: $V_u/2$

Shear resistance: $\frac{1.08}{2} = 0.54$ tons.

Allowable skin friction of the soil: $0.54/3 = 0.18$ tons.

(b) The bearing power of reddish brown sandy clay is 0.71 tons.

Skin friction against pile: 0.18 tons.

Capacity of pile group as suggested: Method I

Surface perimeter of piles for ft. $16 \times 4 \dots 64$ sft.

Total shear for the top 8 ft: $8 \times 64 \times 0.18 \dots 92.17$ T

Total shear for the bottom 7 ft: $7 \times 64 \times 0.18 \dots 80.65$ T

Safe base load: Bearing capacity $\times$ perimeter of the cluster of piles: $0.71 \times (27.75 \times 4.75) \dots 93.58$ T

Total safe group load $\dots 266.40$ T

This is higher than the design load of 160 T and may be adopted:

Total load: $P = 10 \times 16; 160$ Tons $P = B.A. + C. S.$

B: bearing capacity of the soil: 0.71 tons/sft.

A: surface area of pile group: $4.75 \times 27.75 = 131.8$ sft.

C: outer surface of pile group: $2(4.75 + 27.75) \times 15 = 975$ sft.

S: skin friction: 0.18 tons/sft. (A second factor of safety of 3 is adopted for bearing capacity of soil)

Therefore $P: 0.71/3 \times 131.8 + 975 \times 0.18 = 31.19 + 175.5 = 206.69$ (or) 207 tons.

This is just higher than the design load of 160 tons and may be adopted.

The pile has to be designed for 3 conditions:

- (1) Load coming on pile.
- (2) Load during driving.
- (3) Handling stresses.

(1) The load coming on pile is 10 Tons.

The size chosen for the pile is $12'' \times 12''$ and the pile will be $15' 6''$ as required by the above design including anchoring depth:

Stress in concrete: $10 \times 2240/9 \times 9 = 276.5$ lbs./sq. inch. which is safe.

(2) Since the pile may be lifted by 2 point method during the curing period and 1 point method during driving preliminarily these two cases are analysed.

Maximum Bending Moment for two point support:

$M = WL/46.63$ $W = 15.5 \times 150 = 2325$ lbs.

$L = 15.5 \times 12 = 186$ inches.

$= 2325 \times 186/16.63 = 9272$ in lbs.

Maximum Bending Moment for single point lifting:

$M = WL/23.4 = 2325 \times 186/23.4 = 18480$ in lbs.

Value of lateral reinforcement: 0.4%

$= 12 \times 12 \times 12 \times 0.4/100 = 6.918$ cu. inches.

Length of lateral: 36 inches.

Assuming a spacing of 6 inches, total length: $2 \times 36 = 72$ inches.

Area of lateral rod: $6.918/72 = 0.10$ sq. inch.

Hence $3/8''$ diameter lateral at 6" can be used.

For the end two feet of pile on either side the spacing of 3 inches may be adopted.

Cast iron struts will be provided at 3 ft. intervals to prevent reinforcements closing in while concreting. The piles will be provided with drilled cast iron shoes with M. S. straps $2'' \times 3/8''$ cast in as shown below. R. C. C. pile shoe itself can be used in the closer ties.

Pile Design Detail

(To be substituted for a portion of original Design)

Maximum Bending Moment: 18480 in lbs.

Case I. $K: 0.357$

$Kd: 0.357 \times 9.875 = 3.526''$ $J = 0.88$

$Jd: 0.88 \times 9.875 = 8.690''$

Assuming $f_c: 750$ lbs./sq. inch. $f_s = 18000$ lbs. per sq. in.

Area of steel required: $18,480/18000 \times .88 \times 9.875 = 0.1181$ sq. in

However nominal reinforcement of $1/4''$ diameter rods are provided case ii. When one diagonal is horizontal during lifting.

Assuming a first approximation the position of N. A. to be 3.15 inches from the top corner. The distance will be verified by taking moments about C. L. of pile of effective transformed areas in terms of concrete.

Particulars	Area 8	Lever Arm	Moment.
Concrete : $\frac{1}{2} \times 6.3 \times 3.15$...	9.92	6.4	63.5
Steel $15 \times .196$...	2.94	4.8	14.1
Steel $15 \times 2 \times .196$...	5.88	0	0
Steel $15 \times .196$...	2.94	- 4.8	- 14.1
Total ...	21.68	6.4	63.5

Distance of N. A from the centre of pile : $63.5/21.68 = 2.93$ inches.

Since the difference is small this may be taken as correct. Moment of inertia of transformed section about N. A. :

$1/12 bd^3 = 1/12 \times 5.86 \times 2.93^3$...	12
2.44×0.77^3 ...	2
5.88×5.5^3 ...	178
2.94×10.3^3 ...	312
Total ...	504 inch.

Modulus of section for steel : $504/15 \times 10.3 = 3.262^3$ in Behind moment :
 $18000 \times 3.262 = 58716$ in lbs. Hence safe.

Design of capping slab

Total load from pier : 133 tons.

Load per ft. run : $133/15 \times 2 = 4.44$ or say 4.5 tons.

Load on each pile : 8.5 tons.

Load per sq. ft. of pile cap : $4.5/4.5 = 1$

Bending Moment due to piles $2 \times 8.5 (8 \times 3.75) \times 2240 \times 12$
 $= 1,37,10,000$ in lbs.

Bending Moment due to pier load : $4.5 \times \frac{14.75 \times 14.75}{2} \times 2240 \times 12 = 1,31,60,000$ in lbs.

Net Bending Moment : 5,50,000 in lbs.

Bending Moment on AA : due to piles : $2 \times 8.5 (6 \times 3.75) \times 2240 \times 12$
 $= 1,02,82,500$ in lbs.

due to pier loads : $4.5 (13 \times 13)/2 \times 2240 \times 12 = 1,02,20,000$ in lbs.

Net bending moment : 62,500 in lbs.

Hence the greater of the two i.e. 5,50,000 in lbs. is adopted :

Transverse Bending Moment : $Wl^2/8 : \frac{1 \times 2.75^2}{8} \times 12 \times 2240$
 $= 25400$ in lbs.

Effective depth of slab required : $550,000/126 \times 69 = 7.952$ inches.

Depth of slab required from punching shear consideration under piers ;

$$4.5 \times 2240/24 \times 75 = 5.599 \text{ inches.}$$

do. above pile $10 \times 2240/48 \times 75 = 6.322$ inches.

Hence a total depth of 12 inches will be sufficient.

Main reinforcement : $5,50,000/5.75 \times 18000 \times .87 \times 7.95 = 0.7683$ inches.

$\frac{1}{2}$ " rod at 6" centres give an area of 0.883 sq. inches.

Transverse reinforcements : $25,400/18000 \times .87 \times 7 = 0.2317$ sq. inches.

The depth of pile cap may be kept 12" with a levelling course of 6" thick cement concrete :

capping slab under abutment :

Live load ($\frac{1}{2}$ span) 19 Tons.

Dead load $\frac{1}{2}$ spans $31/2 = 15.5$ tons.

Weight of abutment

Bottom 1.41 ft : $27.0 \times 5.7 \times 1.41 \times 140/2240$... 13.69

Next 1.25 ft. $25.0 \times (5.9 + 4.6)/2 \times 1.25 \times 140/2240$... 40.02

Next 8.34 ft. $25.0 \times 0.104 \times 140/2240$... 41.54

Backing course $25.0 \times 0.104 \times 1 \times 140/2240$... 1.36

66.61 say 67 tons.

Total down ward weight : $19 + 15.5 + 67 = 101.5$ tons. say 102 tons.

Load per foot width. $102/27 ; 3.878$ tons.

Load per sq. ft. $3.878/5.75 = 0.6744$ tons.

Load per pile $102/16 : 6.375$ tons.

Bending moment due to piles : $2 \times 6.375 (8 \times 3.75) \times 2240 \times 12$
 $= 10330000$ inch lbs.

due to abutment $3.878 \times 13.5 \times 13.5/2 \times 2240 \times 12 : 9497000$ in lbs.

Net Bending moment : 8,33,000 in lbs.

Transverse Bending Moment : $Wl^2/8 : 6744 \times 2.75 \times 1 \times 2240/8$
 $= 17,250$ lbs.

Effective depth $\sqrt{8,33,000/126 \times 29} = 9.788$ inches.

Steel required : $8,33,000/5.75 \times 18000 \times .87 \times 9.788 = 0.945$ sq. inches.

$3/4$ " rods at 5" centers give an area of 1.05 sq. inches and hence may be adopted.

Steel in transverse direction : $17250/18000 \times .87 \times 9 = 1224$ sq. inches.

20% of main rods area $1.05/5 = 0.201$ sq. inch.

$3/8$ " rods at 6" centres give an area of 0.22 sq. inches. and hence may be adopted.

The total depth of pile cap may be 12" A layer of thick c.c. 1 : 2 : 4 may be laid below to act as levelling course and for anchoring piles.

Observations made during Driving of Test Pile.

Test Pile No. 2 without shoe :

Total length 20' 2" or 20' 0"

Date of driving : 12-1-'57.

Ground Level : 92.50

Name of the work : Constructing a Bridge at mile 12/6 of Ponneri Pulicat Road.

Reduced Level	Depth of Sinking	No. of Blows	Hr. of fall	Average penetration per Blow	Nature of soil pierced through	Remarks
From	To					
92.05	91.25	91"	11"	2' 0"	Fine sand	
91.25	90.75	6"	6"	1"		
90.75	89.00	21"	18"	1 1/6"		
89.00	88.75	3"	2"	1 1/6"		
88.75	87.75	12"	2"	6"		
87.75	86.75	12"	2"	6"	Dark-Grey Sandy-Clay	
86.75	86.25	6"	4"	1 1/2"		
86.25	85.75	6"	7"	6/7"		
85.75	84.75	12"	2"	6"		
84.75	83.75	12"	2"	6"		
83.75	83.25	6"	2"	3"		
83.25	82.25	12"	2"	6"		
82.25	81.50	9"	2"	4 1/2"		
81.50	80.75	9"	3"	3"		
80.75	79.75	12"	3"	4"		
79.75	79.25	6"	2"	3"		
79.25	78.25	12"	6"	2"		
78.25	77.25	12"	8"	1 1/2"		
77.25	77.00	3"	2"	1 1/2"		
77.00	76.50	6"	3"	2"		
76.50	76.00	6"	6"	1"		
76.00	75.00	12"	5"	2 1/2"		
75.00	74.50	6"	3"	2"		
74.50	74.25	3"	2"	2' 0"	Reddish brown Sandy-clay	
74.25	73.75	6"	3"	1 1/2"		
73.75	73.50	3"	2"	1 1/2"		
73.50	73.25	3"	4"	2' 0"		
73.25	72.25	12"	48"	1 1/3"		
72.25	71.25	12"	33"	1 1/3"		
71.25	70.25	12"	42"	1 1/3"		
70.25	69.25	12"	51"	1 1/3"		
69.25	68.25	12"	62"	1 1/3"		
68.25	67.50	9"	54"	1 1/6"		

Results of Observations made while driving Test Pile in the work of Constructing a Bridge in Mile 13/6 of Ponneri Pulicat Road.

Weight of Hammer : 1 Ton.

Length of Test Pile : 20' 0"

Weight of Pile 3000 lbs.

Serial No. of strokes	Reading after the last stroke	Total Settlement	Average Penetration for each blow in inches S.	Height of Drop H.	Remarks.
Date 28-2-1957					
0	0'-10 1/4"	0' 0"	0	0' 0"	R.L. 91.00
1-10	1'-4"	0'-5 1/4"	0-5	1'-0"	
11-20	1'-10"	0'-6"	0-6	2'-0"	
21-30	2'-2 1/4"	0'-4 1/4"	0-45	2'-6"	
31-40	2'-7"	0'-4 1/4"	0-45	2'-6"	
41-50	3'-1"	0'-5 1/4"	0-55	2'-6"	
51-60	3'-10 1/4"	0'-10"	1-0	3'-0"	
61-70	5'-7"	1'-8 1/4"	2-05	3'-0"	
71-80	8'-9"	3'-2"	3-8	3'-0"	
81-90	12'-0"	3'-3"	3-9	3'-0"	
91-100	14'-9"	2'-9"	3-3	3'-0"	
101-110	16'-10"	2'-1"	2-5	4'-6"	
110-120	19'-0"	2'-2"	2-6	4'-0"	
121-134	19'-9"	0'-9"	0-64	4'-0"	
135-140	20'-2 1/4"	0'-5 1/4"	0-91	4'-0"	
141-150	21'-1 1/4"	0'-11	1-1	4'-0"	R. L 67.50
151-160	21'-11"	0'-9 1/4"	0.95	4'-0"	
161-170	22'-7"	0'-8"	0-80	4'-0"	
171-180	23'-3"	0'-8"	0-80	4'-0"	
181-184	23'-6"	0'-3"	0-75	4'-0"	
185-190	23'-9 1/4"	0'-3 1/4"	0-6	4'-0"	
191-195	24'-0"	0'-2 1/4"	0-5	4'-0"	
196-200	24'-2 1/4"	0'-2 1/4"	0-5	4'-0"	
201-205	24'-5"	0'-2 1/4"	0-5	4'-0"	

N.B.—For an average penetration of 1" per blow and a drop of 4' 0" the safe load as deduced from the empirical formula is 7 Tons. The Test pile was subjected to a load of 27.2 Tons and a settlement of 1.8" was observed after an interval of 48 hours. As the settlement is more than the permissible limit, further driving was continued on 10-3-1957.

Results of Observations made while driving Test Pile in the work of Constructing a Bridge in Mile 13/6 of Ponneri Pulicat Road.

Weight of Hammer: 1 Ton.

Length of Test Pile: 20' 0"

Weight of Pile 3000 lbs.

Serial No. of strokes	Reading after the last stroke	Total settlement	Average Penetration for each blow in inches S.	Height of Drop H.	Remarks
Date 10-3-1957					
0	24-11 3/8"	0'-0"	0	0'-0"	R. L. of the L. iron placed against the pile face for reading the scale marked in it while being driven.
1-10	25'-1 1/4"	0'-1 7/8"	0.183	4'-0"	
11-20	25'-4"	0'-2 1/4"	0.275	4'-0"	
21-30	25'-6 1/4"	0'-2 1/2"	0.225	4'-0"	
31-40	25'-8 1/4"	0'-2 1/2"	0.25	4'-0"	
41-50	25'-11 1/4"	0'-2 1/4"	0.275	4'-0"	
51-60	26'-2"	0'-2 1/4"	0.275	4'-0"	
61-70	26'-4 1/4"	0'-2 1/4"	0.225	4'-0"	
71-80	26'-7 1/4"	0'-3"	0.30	4'-0"	
81-90	26'-9 1/4"	0'-2 1/4"	0.225	4'-0"	
91-100	27'-0"	0'-2 1/4"	0.25	4'-0"	
101-110	27'-2 1/4"	0'-2 1/4"	0.25	4'-0"	
111-120	27'-4 1/4"	0'-1 1/4"	0.175	4'-0"	
121-130	27'-6 1/4"	0'-2"	0.20	4'-0"	
131-140	27'-8"	0'-1 1/4"	0.175	4'-0"	
141-160	27'-10 1/4"	0'-2 1/4"	0.125	4'-0"	
161-170	28'-1"	0'-2 1/4"	0.25	4'-0"	
171-180	28'-3"	0'-2"	0.20	4'-0"	R. L. on top of follower after it had received the last stroke 63.50
181-190	28'-5 1/4"	0'-2 1/4"	0.225	4'-0"	
191-200	28'-7 1/4"	0'-2"	0.20	4'-0"	
201-210	28'-9 1/4"	0'-2 1/4"	0.225	4'-0"	

N.B.—The rate of settlement is seen to be steady with no appreciable variation. For purpose of evaluating the safe load, the average penetration per blow is taken as 0.2"

$$P = \frac{2wh}{S \left\{ 1 + \frac{wP}{W} \right\}}$$

$$= \frac{2 \times 2240 \times 4}{0.2 \left\{ 1 + \frac{3000}{2240} \right\}} = \frac{3 \times 2240}{0.2 \times 2.34} \times \frac{1}{2240}$$

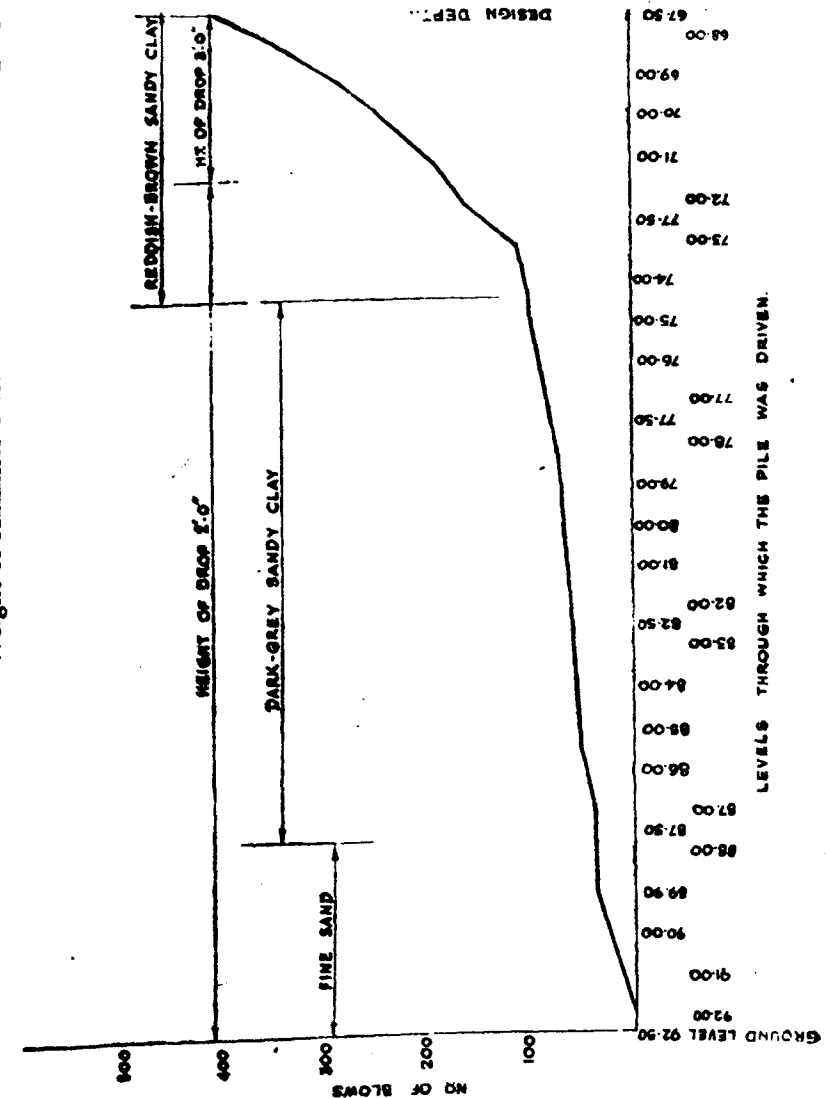
$$8/468 = 17.1 \text{ or } 17 \text{ Tons.}$$

P Safe load.
W Weight of hammer 1 ton.
H Free fall 4' 0"
WP Weight of pile in pounds
3000 lbs. S Penetration 0.2"

ANNEXURE - D
Date of Driving
12-1-57.

Test Pile No. 2 (without shoe).
Length of Pile 20' - 0".
Weight of Hammer 1 ton.

Constructing a Bridge in Mile 12/6
of Ponneri - Pulicat Road



NO. OF BLOWS

500
400
300
200
100
0

HEIGHT OF DROP: 2.0
2.0
2.6
3.5
4.0

HEIGHT OF DROP 3.5
HEIGHT OF DROP 4.0

FINE SAND
DARK GREY CLAY
SAND WITH SMALL % STIFF CLAY
REDDISH BROWN SANDY CLAY
NO SETTLEMENT UNDER A TEST LOAD OF 27.2 TONS.

RATE OF DRIVING 2.57

DATE OF DRIVING 10-1-51

SETTLEMENT UNDER TEST LOAD OF 27.2 TONS.

PENETRATION WHILE DRIVING

30.0
27.0
24.0
21.0
18.0
15.0
12.0
9.0
6.0
3.0
0

GROUND LEVEL 0

Construction of the Bridge across the Kaduvayar near Nagapattinam

BY

Sri K. S. LOGAVINAYAGAM, B.E.,

Divisional Engineer. (H) Nagapattinam.

Synopsis

(The Bridge across the River Kaduvayar near Nagapattinam consists of 14 spans of 15' 11" each, two spans of 15' 5", two end spans of 14' 11" and a lift span of 31' 10". The superstructure of the bridge consists of simply supported slab 15" thick over pier bents designed for I. R. C. class A loading and 12' 0" clear roadway.

An important feature of the bridge is the provision of lift span, between piers 10 & 11, the first of its kind in Madras State.

The paper describes the design and other details of lift span. The rectification of old wells previously sunk which were utilised for the construction is also described).

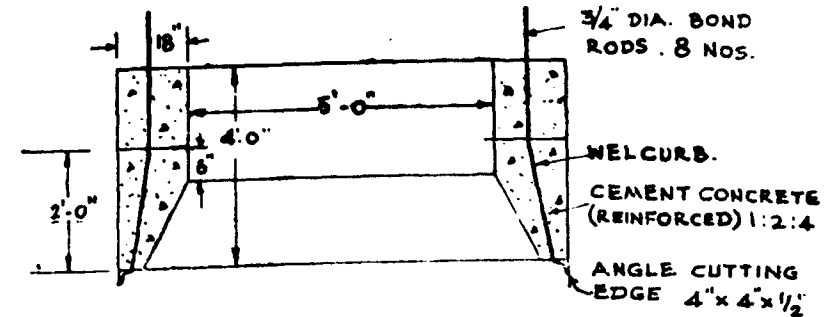
Kaduvayar River and the location of the Bridge

The river Kaduvayar flows to the west of Nagapattinam town separating it from a group of villages along the sea coast spread over to a length of about 5 miles upto Velanganni. The villages were completely cut off from the town especially during floods. Great inconvenience was caused to the villagers as their fish and vegetables remained unsold during the times of floods. To some extent villagers carried their produce by ferries but this was inconvenient and very risky being at the estuary of the river. Therefore as early as 1938, construction of a bridge was proposed. The construction which was actually taken up in 1939 had to be deferred during world war owing to scarcity of construction materials. The work was put on hand again in 1954. The port authorities at Nagapattinam insisted that a lift span should also be provided to accommodate boats and Dredgers to pass through. Hence it was that a lift span was also constructed to provide additional facilities.

During the original construction wells were sunk at 35' 10" centres and abandoned due to war. To economise the cost of construction, it was proposed, naturally, to utilise the abandoned wells. Consequent on the adoption of I.R.C. 'A' class loading, pressure intensity of the foundation soil was found to be inadequate to the onslaught of new loading adopted.

The Well Foundation

So, to cater for the increased loading on the bridge the existing spans were halved and additional new wells were sunk between the old ones. The new wells have reinforced curbs provided with cutting edge of angle iron as shown in the sketch.



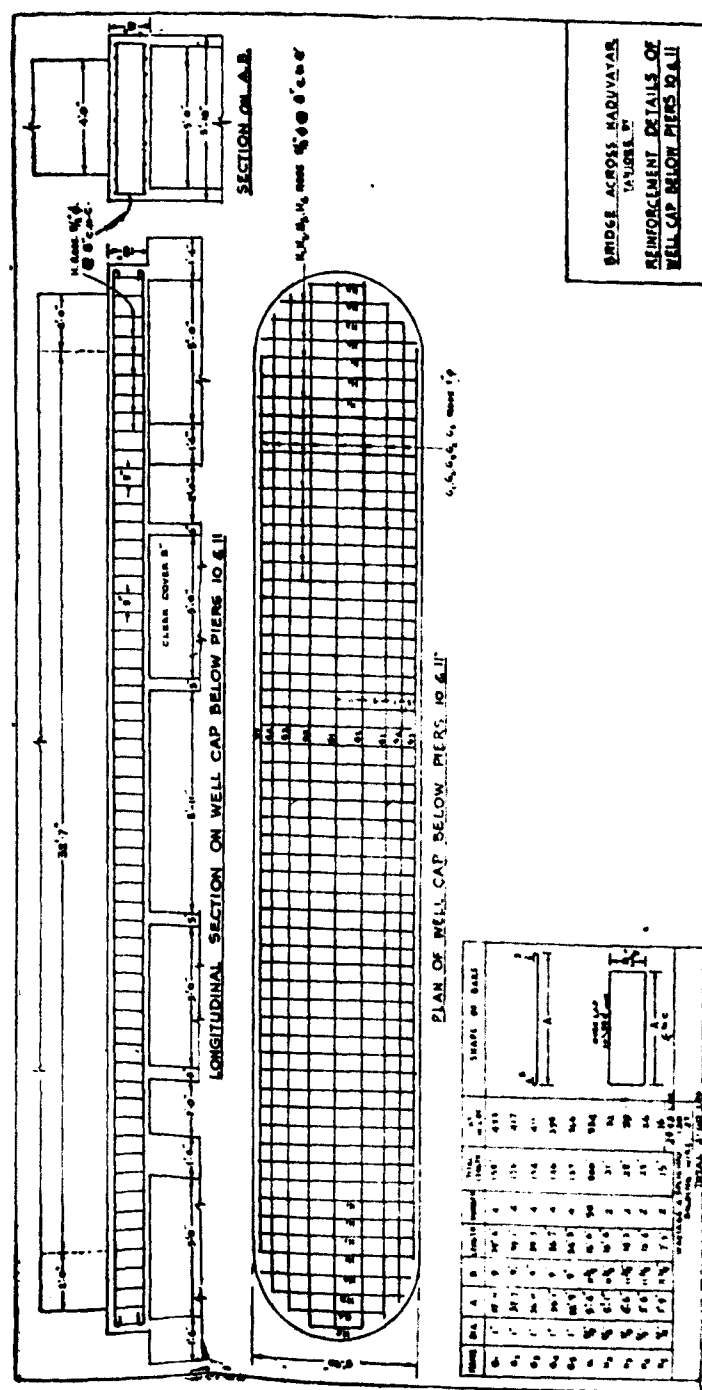
CROSS SECTION OF
A WELL AT BOTTOM.

Fig. 1.

It was proposed to conduct load test on wells under piers 10 or 11 as these structures have to support the lift span. As the kentledge required was of great magnitude it was proposed to take undisturbed soil samples at the bottom level of the well under pier 10 and 11 as near the well as possible and to send the samples for determining the bearing capacity at the Highway Research Station, Guindy.

The soil profile at the site of the bridge consists of silty sand underlain by clay with an admixture of silt and clay. The soils are dark grey in colour and contain no organic matter. The hydrometer analysis has indicated that the soil profile consists of silty sand with high percentage of sand in the higher elevations. In lower elevations the soil contains higher percentage of silt and clay giving low density values. It was also found that the natural moisture content increases as the depth increased.

The soil from the foundation level i.e., at R. L.—30.00 had given an ultimate bearing capacity of 0.77 tons per square foot, and 0.75 tons per square foot for the 10th and 11th piers respectively. The net intensity of pressure on soil at the foundation elevation (—30.00) had been worked out to be 0.79 and 1.18 tons per square foot under new and old well respectively, adopting a factor of safety of 3. But it may be pointed out here that the dead load of the pier is considerably large being about 166 tons and the live load is 27 tons. Hence the settlement corresponding to load transmitted to the soil at the base of the bridge pier, due largely to the dead load, with the progress of construction should have been completed gradually with the construction of the structure and so further settlement may not be appreciable.



However levels are being touched on the top of piers to ascertain any settlement in the structures.

Pluggings and Capping Slab

The new wells having a greater thickness of steining have been filled with sand for intermediate filling while the old ones were plugged with vitrified brick jelly concrete (Mix. 1:3:7½) in cement mortar between top and bottom pluggings. The bottom plugging consisted of Cement concrete 1:2:4 in 1½" granite metal for 4' depth, and the top plugging with cement concrete 1:4:8 for 2'. The capping slab over old wells have a width of 5—10" (the external diameter of old circular well) while those over new ones have a width of 8". The capping slab over wells for lift span piers have a width of only 5—10" the thickness being 22". This slab was redesigned for additional load coming over them due to change of brick masonry for the pier to plum concrete in cement mortar (mix. 1:3:6) with 20% plums of 6" granite cubes.

Substructure

To minimise the dead weight coming over the foundation, the usual massive piers have been substituted by a portal frame, in the shape of two circular columns, 2' diameter and cross beam of size 3' × 2' over the columns. The construction of these portal frames, otherwise called bents have proved lighter and execution was also easy and expeditious. The abutments are of brick masonry in Cement mortar.

The Super Structure

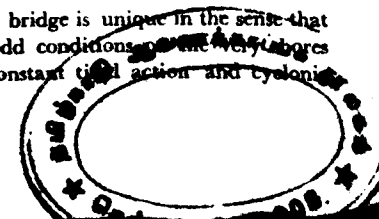
The superstructure consists of simply supported R. C. C. slabs 17"—11" long and 1—3" thick with a clear span of 15"—11" The hand rail posts are of size 6" × 7" × 2—7" The hand rails consist of 4 Nos. 1½" Dia. G. I. pipes. Provision has been made for the expansion of deck slabs by applying craft paper over the top of the piers before laying concrete for smooth sliding. Further a 1/2" gap is given between adjacent deck slabs. The gap has been filled with a mixture of chopped hump, bitumen sand and cement in a hot condition. The mastic for premoulded joint filler composed of the following ingredients:

Very fine clean sand	...	by wt. 60%
Bitumen R-25	...	.. 30%
Cement	...	.. 8%
Chopped hump	...	.. 2%

The roadway over the bridge provides for 12' clear passage between wheel guards of size 1—6" × 1—0".

The Execution Difficulties

Besides the provision of a lift span the bridge is unique in the sense that the construction had to be done all along in odd conditions on the very shores of the Bay of Bengal, being subjected to constant tidal action and cyclonic weather in winters.



Weather Conditions

The bridge being in the vicinity of Sea, special precautions had to be taken in the construction for extra cover for all reinforcing steel to guard against saline action.

Rectification of Old Work

In all stages of the work many difficulties were met with and had to be tackled promptly and precisely. One of such things was the rectification of the old works. Most of the old wells sunk over 15 years ago, were out of alignment with much cant. The position of the wells should have been disturbed due to scouring. The well in down stream side in pier 10 was out of alignment for about 4-6". The well was silted inside and embedded in earth for about 22 feet when the island was formed. It took about 20 days to bring the well to its correct position.

The Operation

The silt inside the well and the earth under the cutting edge on the higher side was removed to minimise the dead load of the well and the reaction at the bottom of the well respectively.

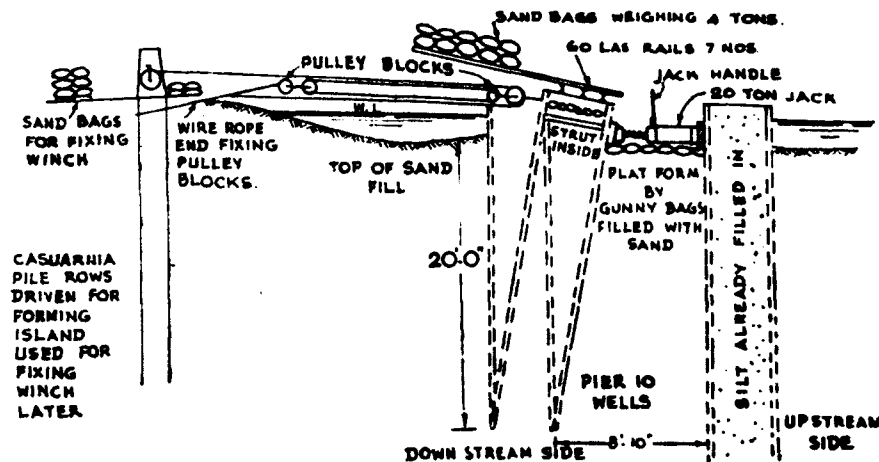


Fig. 2

By a Horizontal Pull

The top of the well was tied by a loop with a stout ship chain of 1" links and this was connected to a two ton winch by a compound pully system as shown in the figure. By this arrangement the mechanical advantage of the system was considerably increased.

Eccentric Loading

Further the well steining having heavy reinforcement was broken to a depth of 1-6" around (which was useful afterwards for further steining) and the reinforcement was used for anchoring and fixing 60 lbs. rails as shown in the sketch. These longitudinal rails were again tied to cross rails by means of 1/4" M. S. rods for stability. Over one end of the longitudinal rails sand bags 100 Nos. were loaded to give a net cantilever action at 5' away from the higher end of the steining (i.e., this arrangement gave a turning moment of $100/25 \times 50$: 200' tons.

Jack Arrangements

Apart from the above, the existing site conditions were fully utilised for advantage. The adjoining well on the upstream side was already filled with silt. This was used as a big struct backing vertically fixed to support a jack of 20 ton capacity horizontally arranged as illustrated. Necessary precautions were taken not to disturb this well which was in its correct position. Only a part of the load necessary to turn a hollow well under operation was transmitted to this well through the Jack.

Air Jet

An air jet with 3/4" G. I. pipe 25' long connected with a rubber hose to an air compressor of capacity 5 cft. at a maximum pressure of 200 lbs. was also operated. This air jet was applied very near the well steining on the higher end to disturb the earth to give less skin friction.

To impart more strength to the hollow well with a very thin shell, cross struts, just reacting the pressures coming over the steining, were fixed to avoid any cracking due to the undue stresses.

After making all the above arrangements air jet was first applied for about five minutes to disturb the soil around the well. Then it was applied on the higher side. At this juncture the winch and the jack were operated simultaneously and this well began to move to its original position. The jack of 20 ton capacity and the two ton winch with a pulley system gave a force of 35 tons nearly for the rectification of the cant. The effect of the air jet could not be measured, but it silently helped to avoid any damage to the well. It was moved by 5" to 6" daily and finally it was brought to the correct alignment in a safe condition.

Tidal Condition

A natural difficulty in the course of the work was the tidal condition of the river, restricting the working hours to practically half a day every day. The tidal action had a serious detrimental effect on the islands formed for the foundation work.

Formation of Islands

About one fifth of the cost of construction of the bridge had to be spent for the formation of islands for well foundation and casting of capping slab. Casurina piles 4" to 5" Dia. had been procured and driven in two rows to an average depth of about 10 feet. The depth driven depended upon the depth of earth filling required for each island. Generally casurina piles 20 feet only and 4" to 5" dia. driven at one foot centres to an average depth of 10 feet in two rows spaced 3 feet clear in parallel was found to be sufficient to retain a clay fill of 15 to 18 feet for about four months.

The pile rows were again reinforced by means of longitudinal walings 2" to 3" dia. in two or three rows, all above or near the low tide level. Sometimes horizontal ties were also provided when the rows of piles were nearest, where the earth pressure was likely to be heavy. To give a reasonable protection to the tidal and wave actions on the islands formed inside the pile rows casurina frame work with palmyrah leaves were fixed. Gunny bags filled with clay were arranged between the pile rows to retain the clay to be filled in the area surrounded by pile rows. Clay had to be conveyed from a long distance and dumped into islands. The top of the islands were fixed at R. L. 1.50 just above the normal high tide level before and after the winter season.

Another difficulty encountered during the execution of the work was the formation of islands from pier 11 to pier 13. This portion was deeply scoured and hence specially long piles from 25' to 30' were to be procured and driven to enough depths in deep waters using boats and catamarans. Further earth filling by boats had also taken a considerable time for completion.

Erection of Fallen Wells in Pier 11

During the heavy floods that swept on the 19th September 1956, the two old wells under pier 11, had practically fallen down horizontally on the bed with a depth of water about 18 feet, on account of heavy scour.

The wells were filled with silt and were weighing about 35 tons each. The erection of these wells in position with special derricks and skilled manipulation should be specially treated as an achievement.

There was a great difficulty in procuring suitable materials such as derricks, wire ropes, chain blocks etc. for the erection of the fallen wells. Stout and long wooden posts were considered suitable in this case, because they could give greater bearing area unlike steel derricks which are generally heavy for the section. Three casurina derricks 12" to 15" diameter and about 50 feet long, a 20 ton chain block and a set of erection materials such as wire ropes ranging from 3/8" to 1 1/4" dia. shackles, chains etc. were obtained on hire. Other tools used were a 2 ton winch, a 10 ton winch and three boats.

Before the erection of the wells was commenced the exact location and position of wells were found out by expert sinkers and plotted on a paper.

On the paper the exact position of the legs of the derricks was also determined by calculation so that the resultant would fall within the base of the tripod. The derricks were erected for lift one of the fallen wells in the form of a tripod. The 20 ton chain block was hung from the top of the tripod. Further a pulley block was also hung from the top to supplement the chain block. A loop formed of a wire rope 1 1/4" dia. was hung from the chain block and the pulley block.

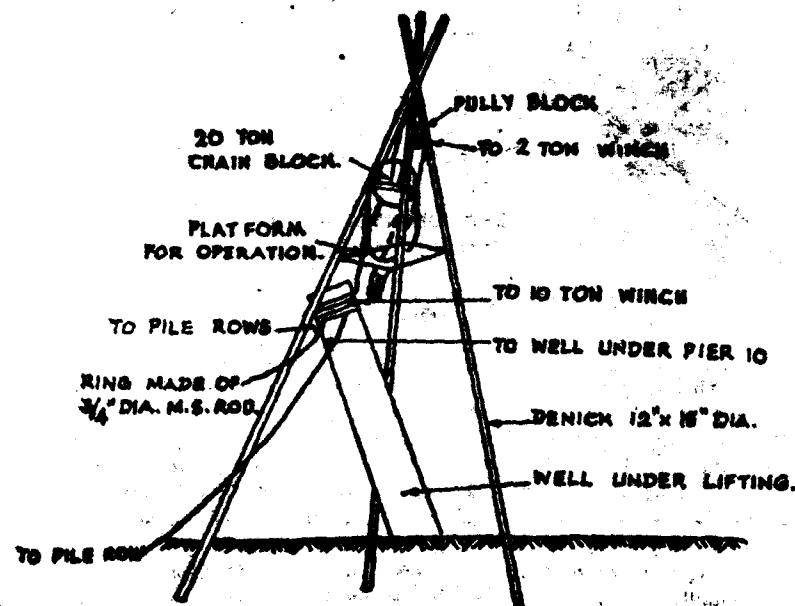


Fig. 3

The loop was attached to the top end of the well. The chain block was operated from a platform formed at an intermediate height of the tripod just above the high tide level. The pulley block was wound with wire ropes for greater mechanical advantage and was attached to a 2 ton winch, supplementing the chain block in lifting the well. Further another loop was formed around the top of the well which was attached to a 10 ton winch fitted with a pulley block system. A stout wire rope was wound round the well and attached to a well of the Pier 10 and kept taut as the well was being raised to keep it in the raised position. A ring made of 3/4" dia. rod in the shape shown below was also attached to the top portion of the well to accommodate two wire ropes which passed through a set of couple of loops on either side to keep the well without again falling down to the bed by anchoring the wing to the casurina pile posts.

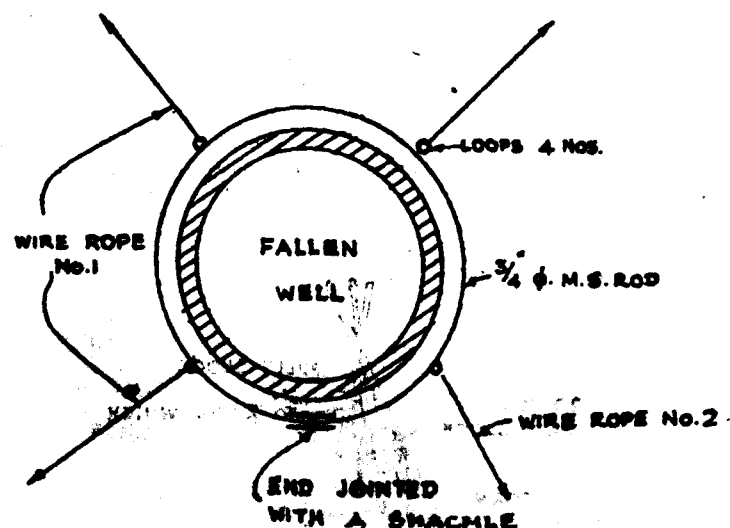


Fig. 4.

When the well was lifted the wire ropes were tightened or loosened as the case might be. By gradual rise the well was brought to a steeply inclined position. To ensure proper grip on the well in question when the top of the well was seen above the low tide level for about one foot four holes were made diametrically in the steining at the top*. These holes were passed through with $1\frac{1}{4}$ " dia. steel wire rope was passed through the holes in the form of rings and the rope rings was attached to the 20 Ton chain block*. The well was vertically lifted by the above mentioned wire rope hangers for about 6" from the bed and horizontally pulled in proper directions by means of winches and wire rope and placed in position.

Fixing of Erected wells

Casurina piles 5" to 6" dia. 25 to 30 feet long were driven along the inner side of the well using the tripod. The top of the piles were kept at the same level and they are connected by horizontal struts diametrically. This arrangement gave a good support to the well steining in the running water. The erection of the other well also was done in the same way. When once the island around these wells were formed the piles driven in the wells were pulled out for further addition of steining and sinking.

When the wells under piers 10 and 11 were sunk fully the gap between the old wells measured longitudinally only 29 feet instead of 30 feet. The reasons for this might be due to the following facts.



Original view of the Old Wells

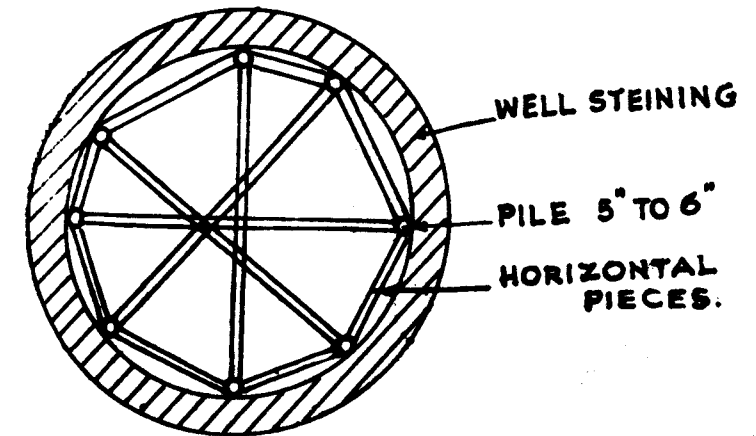


Fig. 5

The adjacent portion of the river near the island for pier 10 had been scoured deeply to about -20'00. Due to this the pile rows near the deeper bed collapsed and the earth from the island slipped towards the deeper bed with the earth movement, the two old wells sunk only upto about -20'00 might have moved laterally to some extent. Towards pier 11 a cant of 4" for 30 feet depth was observed when the old wells in pier 10 were fully sunk. Further there might have been some dislocation towards pier 10 when the fallen wells under pier 11 were erected in position. Ultimately there was only 29 feet between the old wells in pier 10 and pier 11 instead of 30 feet.

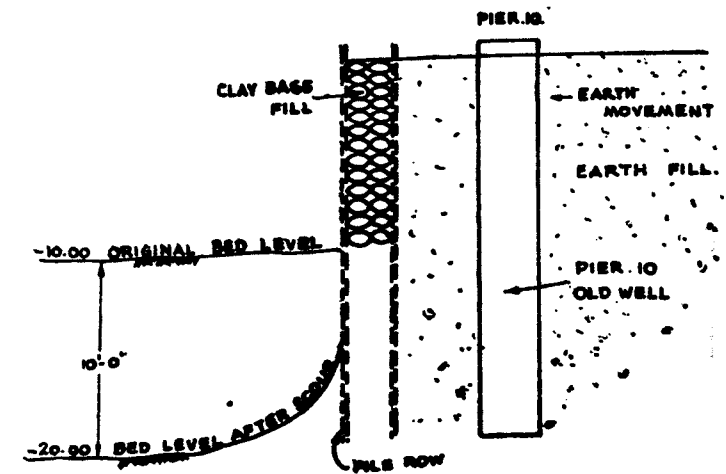


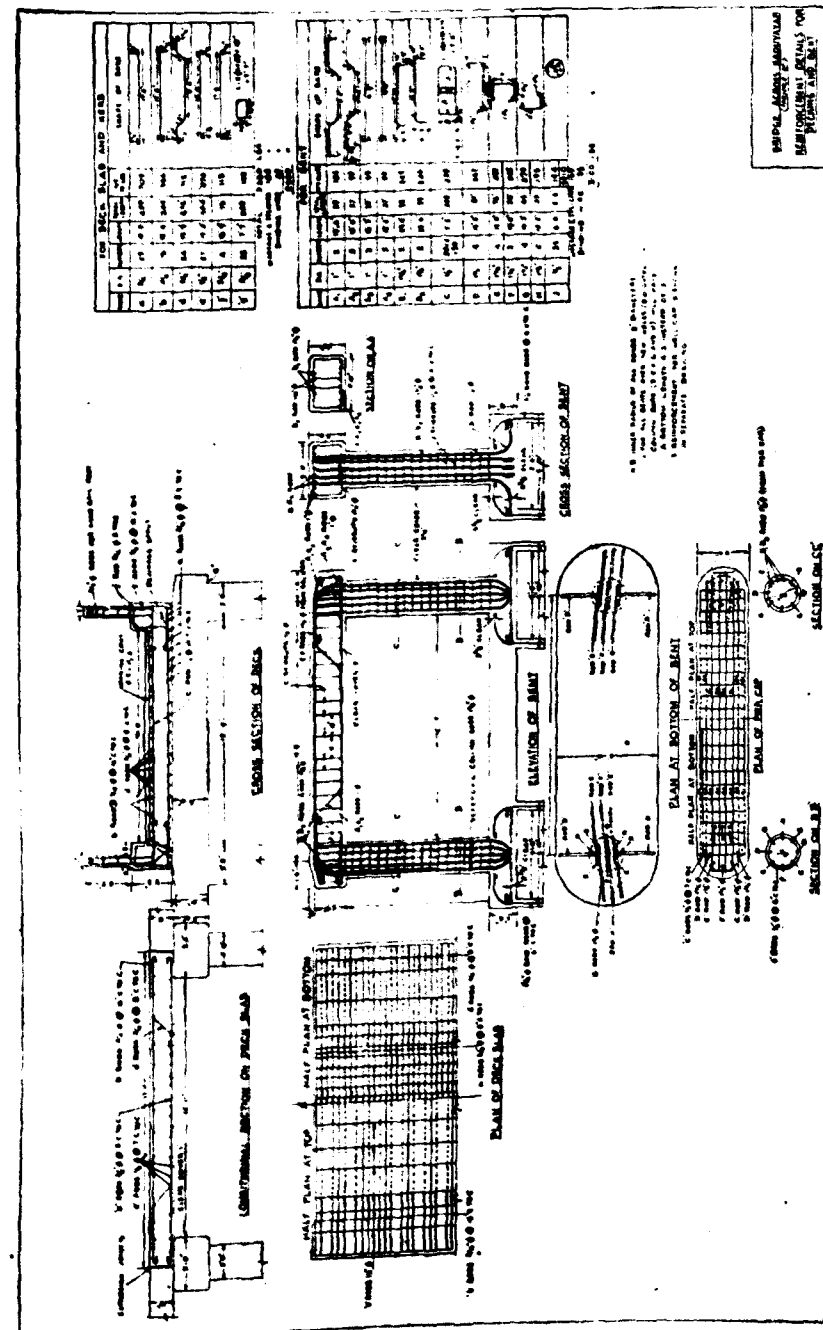
Fig. 6



A View of the Completed Bridge



Another View of the Completed Bridge



9-210-22 ACTIONS, ADVISORY AND
RECOMMENDATION DETAILS FOR

Consequent on the above the pier shape had to be modified. Previously the piers 10 and 11 were designed with a base width of 5'-6" and the top width of 4'-0" with equal batter on either side. Then the pier shape was modified so that the clear span at R. L. -9.25 could be 31'-10" between piers which span length had been adopted for the lift span. The base width hence worked out to 4'-10" the top width being 4'-0". The front sides of the piers were given a batter of 1/12 and the rear sides were kept vertical. The capping slab was also redesigned and the thickness of the slab was also increased to 22" from 18".

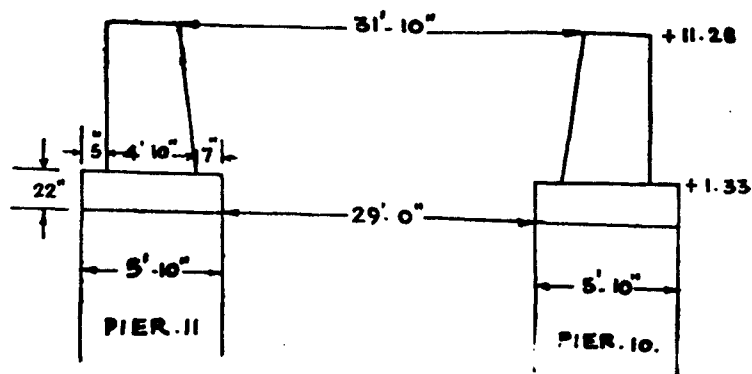


Fig. 7.

The Lift Span

An unique feature in the bridge is the provision of a lift span between piers 10 and 11.

Details of Lift Span

The lift span consists of four columns each made up of 2 mild steel channels $10'' \times 3\frac{1}{2}''$ placed facing each other providing a rectangular enclosure and braced together strongly. The top of columns are connected by overhead trusses. At the top of each column is provided a toothed wheel of 2'-6" diameter about which is suspended the deck and the counter weights on the other side by chains passing over the wheel. The counter weights move up and down outside the tower. The drive for lifting is taken from an electric motor, supported on a platform in the over head trusses. The lift span can also be raised by hand operation in case of failure of power supply. When electrically operated the lift span can be raised fully in 40 seconds and lowered in 35 seconds.

Power Equipment

The power for lifting is obtained from a 3. H. P. A. C. reversible type slip ring Induction motor of 750 R.P.M. which on account of its high initial torque is best suited for operating lift spans.

The Lifting Mechanism

The motor is coupled to a worm gear reduction unit and these two are mounted one over the other on the central platform at the top. The R.P.M. of the motor is reduced by 150 times by the gear unit and the output shaft will rotate at 5 R.P.M. The diameter of the output shaft is 2.5". This shaft which is the main shaft is extending on both sides if the gear unit and flexible couplings are attached to it. The main shaft is supported in the midway by bearings. The main shaft drives two end shafts at right angles by level gearing. The level gearing is of hypoid type and the gear ratio is 35: 10 so the end shafts 3" diameter revolves at 1.43 R.P.M. and are supported by bearings. Both the worm and level reduction units are provided with thrust bearings to take up the thrust developed during the drive in either direction.

Hand Operation

Hand operation is necessary when power supply fails. A detachable hand wheel of 1'-6" radius is provided on the off side of the motor. Assuming that one man can exert a force of 20 lbs. on a lever with a speed of 150 ft./min. developing about 1/10 of H. P. the time required for lifting the span will be approximately 17 minutes for one man operation and 8.5 minutes for two men operation.

Toothed Wheels

The toothed wheels are made of cast steel. The chain sits over the wheel touching only on alternate teeth and hence a slight jerky motion can be observed only when electrically operated.

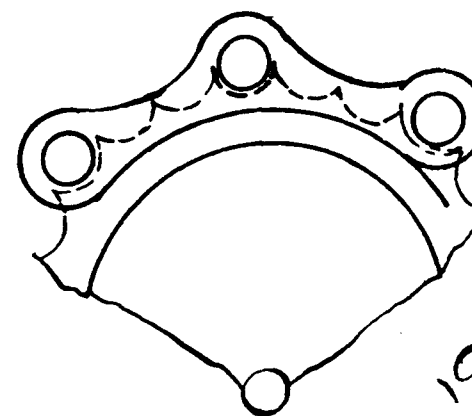
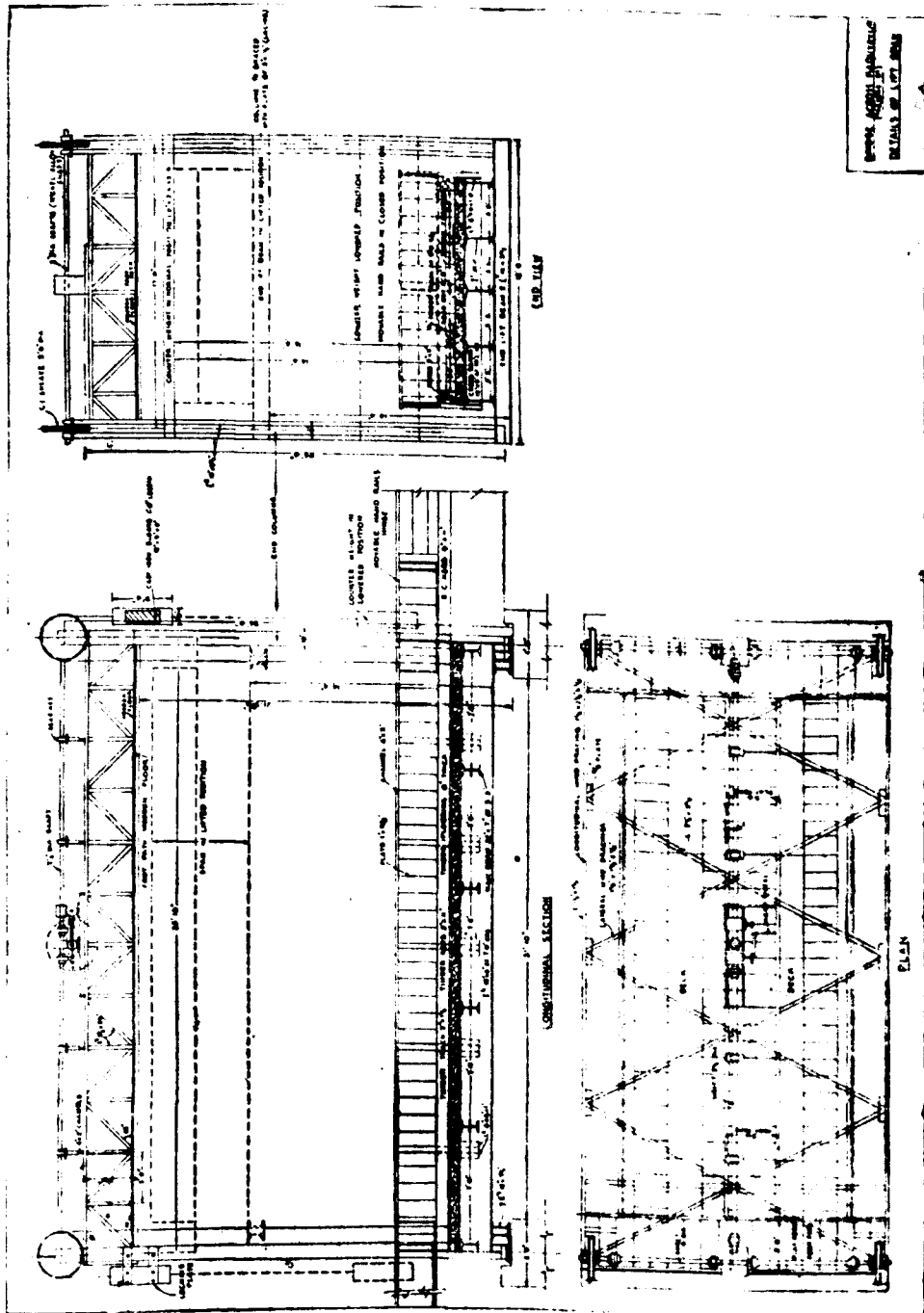


Fig. 8.

Counter Weights

The counter weights are steel boxes of size 18'-0" x 4'-0" x 1'-0" overall, filled with cement concrete 1: 2: 4. Further cast iron balancing weights are placed over the box the entire unit giving a load of 9 tons. When the lift span is



in its raised position, the counter weights barricade the road way in the lowered position. The counter weight is painted in signal red colour for obvious reasons.

Guides

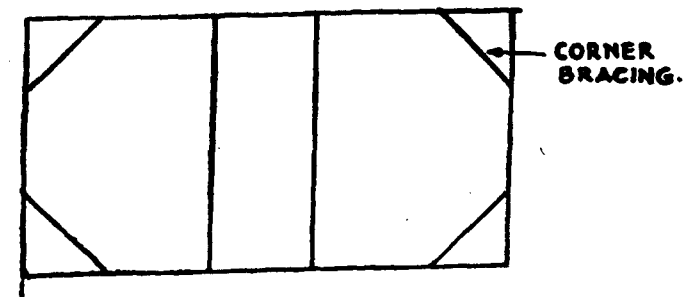
Four guide rollers are fixed at four corners of the lift span, rolling against the web and flanges of each column. The guides are spring mounted to allow for temperature expansion and contraction of the members and to withstand tractional thrust. The counter weights also are provided at both ends with guides which engage trucks on the columns.

Buffers

At the lowered position of the deck the end lift beam comes in contact with buffers facilitating the smooth landing of the deck. The buffer is of synthetic rubber padding of size 14" x 14" and 5 in numbers.

Overhead Trusses

The lateral bracings and longitudinal overhead bracings are of warren type with the members consisting of $2\frac{1}{2}" \times 2\frac{1}{2}" \times \frac{1}{4}"$ M. S. Angles. The horizontal strut is counter braced "N" girder type consisting of $2\frac{1}{2}" \times 2\frac{1}{2}" \times \frac{1}{4}"$ angles, corner bracings to withstand wind pressure due to gale at a speed of 60 Mile per hour was also provided. These bracings are provided to give more rigidity to the corners.



PLAN AT TOP OF LIFT SPAN
SHOWING CORNER BRACINGS.

Fig. 9.

Further overhead trusses carrying platforms and foot paths are provided for motor and operating men. A ladder is provided for operators to climb up.

The Decking

The decking for the lift span consists of five main R. S. joists $22" \times 7"$ spaced at 3'-3" centres, supporting cross girders $12" \times 5"$ at 7'-0" centres. Over the cross girder 6" thick timber flooring with a wearing timber track of size $3'-6" \times 0'-\frac{1}{4}"$ spaced 2'-0" clear at the centre is provided. The main girders rest

on an end lift beam comprising of two R. S. J.'s of size $10'' \times 5''$. The total weight of this part is 18 tons.

Locking device to lock the lift span in position securely before traffic is admitted to the span is provided. A $1''$ thick plate firmly anchored on to the top of the pier and the projecting place has a slot in which a $1''$ pin can be inserted connecting the deck and the locking place.

Locking Arrangements

The counter weights are locked by means of four brackets projecting out from the column under neath the counter weights. The brackets can be turned otherwise when the lift span is to be operated.

Automatic Limit Switches

Automatic limit switches are provided to cut off current at each end of travel of the span and at such a point near the limit of the movement of the span that it can come to halt.

The lift span provides a free board of $6'-8''$ above M.F.L. (-3'60) for the use of small boats. This is capable of a vertical travel of $14'-4''$ thus affording a total clearance of $21'-0''$ for allowing big boats and dreggers.

Approach Roads

The left approach road has been formed mostly by conveyed earth with a side slope of 1:1 revetted with cement concrete slabs with lugs. A mix of 1:4:8 was adopted for the slabs from the point of view of economy. The slabs with richer mix were used for the revetment in bottom layer. The slab revetment is supported by a toe wall in cement concrete 1:3:6 of size $1'-0'' \times 1'-0''$ in section. The slabs are backed with unscreened gravel for $6''$ thick over the slope.

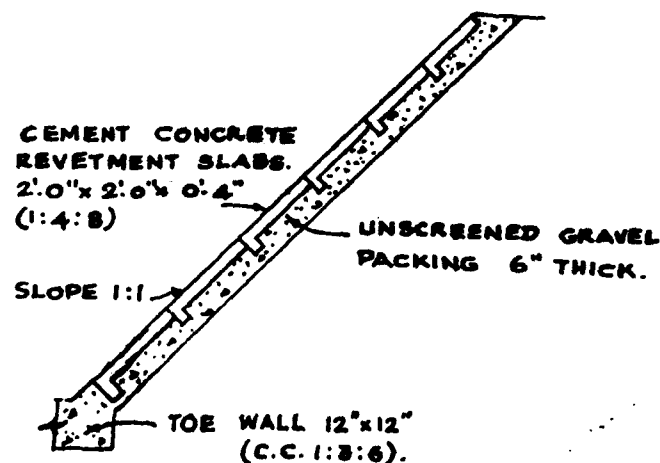


Fig. 10.

The right approach road has two culverts with 6 and 10 rows of $2'-0''$ diameter R. C. C. Pipes to drain water from the sea impinging on the road in winter seasons and to function mostly as balancing culverts.

Consumption of Materials

It may be of interest to give some particulars regarding the consumption of materials in the construction. 425 tons of cement and 75 tons of steel have been used in the bridge in addition to 25 tons of steel and iron and 9 tons of timber consumed in the fabrication of the lift span. The cost of construction of the bridge is Ra. 5,38,000/-

APPENDIX

VERTICAL LIFT SPAN

Design of Flooring :

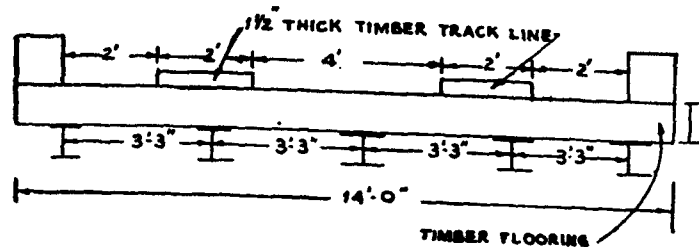


Fig. 1.

Loads (1) Dead Load :

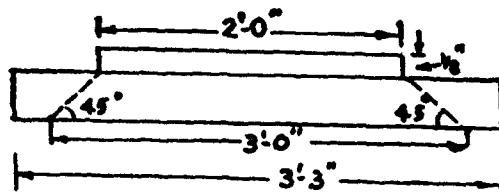


Fig. 2.

$$\text{Self weight} = \frac{6}{12} \times 50 = 25 \text{ lbs./sq. ft.}$$

$$\text{Wearing surface} = \frac{2 \times 1.5}{12} \times 50/3 = 4.17 \text{ sq. ft.}$$

$$\underline{\underline{29.71 \text{ sq. ft.}}}$$

$$\text{Permanent Load factor} = 2$$

$$\text{Dead load Moment} = \frac{WL^2 \times 2}{10} = \frac{2 \times 29.71 \times 3.25 \times 3.25}{10} = 12$$

$$= 739 \text{ or say } 740 \text{ lbs.}$$

(2) Live load :

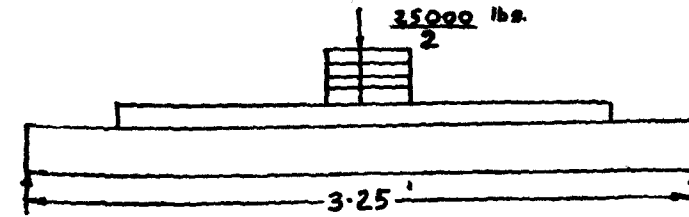


Fig. 3.

$$\text{Max. B. M.} = \frac{W.L.}{4} = \frac{25000}{2} \times \frac{3.25 \times 12}{4} = 121875$$

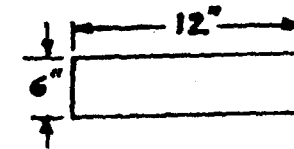


Fig. 4.

Total B. M. due to dead and live loads.

$$740 + 121875 = 122615 \text{ in lbs.}$$

$$\text{Max. Fibre stress} = \frac{M}{Z} = \frac{122615}{1/6 \times 12 \times 6 \times 6} = 1706 \text{ lbs./sq. inch.}$$

∴ This is safe as safe stress for Burma or Malabar teak wood is 2200 lbs./sq. inch.

Design of Main Beam :

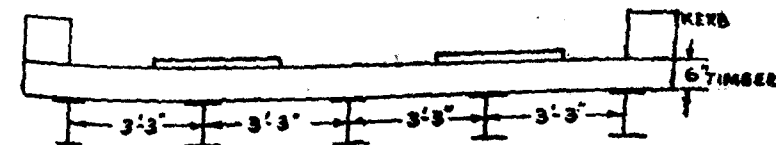


Fig. 5.

5 main beam of 22" × 7" at 3' 3" centres are assumed with a number of Cross beams the load carried by each girder is assumed as 3/4 of wheel load B.M. due to live loads will occur in the position shown below :

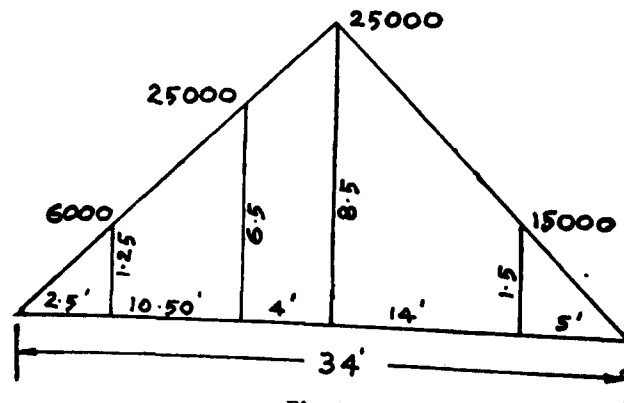


Fig. 6.

$$\begin{aligned}\text{Max. B. M.} &= 6000 \times 1.25 + 25000 \times (6.5 + 1.5) + 15000 \times 1.5 \\ &= 7500 + 3,75,000 + 22,500 \\ &= 4,05,000 \text{ ft. lbs.}\end{aligned}$$

$$\text{Impact} = \frac{30}{45 + L} = \frac{30}{45 + 34} = \frac{30}{79} = 0.38$$

With impact, B. M. due to live load

$$= 1.33 \times 405000 = 4,48,900 \text{ ft. lbs.} = 67,06,800 \text{ in lbs.}$$

$$\text{Fraction of a wheel load to each stringer} = \frac{S}{45} \text{ i.e. } \frac{3.25}{4.5} = 0.7223$$

$$\text{For one main beam} = 0.7223 \times \frac{1}{2} \times 6706800 = 24,23,000 \text{ in lbs.}$$

DEAD LOAD MOMENT

1. Distributed Load:

$$\begin{aligned}\text{Weight of Wearing Surface} &= 2 \times 1/8 \times 50 = 12.5 \text{ lbs.} \\ \text{Weight of decking} &= 25 \times 3.25 = 81.25 \text{ lbs.} \\ \text{Self weight of beam} &= 75.00 \text{ lbs.}\end{aligned}$$

$$\text{Total Wt./sq. ft.} = 168.75$$

$$\text{B. M.} = \frac{WL^2}{8} = \frac{168.75 \times 34 \times 34 \times 12}{8} = 2,92,700 \text{ in lbs.}$$

2. Loads due to Cross Beam:

$$= 12' \times 5' \times 32.$$

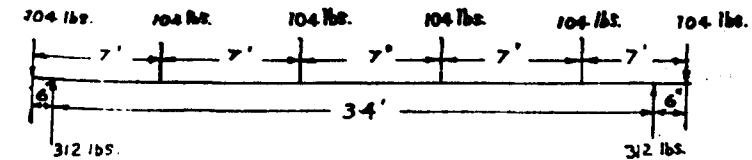


Fig. 7.

$$\text{Each load} = 3.25 \times 32 = 104 \text{ lbs.}$$

$$\begin{aligned}\text{Max. B. M.} &= 312 \times 17 - 104 (17.5 + 10.5 + 3.5) \\ &= 5304 - 104 \times 31.5 \\ &= 5304 - 3276 \\ &= 2033 \text{ ft. lbs.} \\ &= 24336 \text{ in lbs.}\end{aligned}$$

$$\begin{aligned}\text{Total B. M. due to Dead and live loads} &= 24,23,000 + 2,92,700 + 24,336 \\ &= 27,40,036 \text{ in lbs.}\end{aligned}$$

$$Z = \frac{M}{f_s} = \frac{27,40,036}{8 \times 2240} \text{ in}^3 = 153.0$$

$\therefore 22'' \times 7''$ R. S. J. giving $Z = 152.4$ is chosen, which is quite sufficient as the different is only C. S. in 153 which is less than 0.5%.

Design of End Lift Beam:

Dead weight of the whole deck.

$$\begin{aligned}(1) \text{ Weight of timber flooring} &= 34 \times 14 \times 1/3 \times 50 = 12600 \text{ lbs.} \\ (2) \text{ Weight of wearing surface} &= 36 \times 2 \times 1/8 \times 50 \times 2 = 900 \text{ lbs.} \\ (3) \text{ Weight of timber kerbs} &= \frac{36 \times 9 \times 11 \times 50 \times 2}{12 \times 12} = 2475 \text{ lbs.} \\ (4) \text{ Weight of Hand Rails.} \\ \text{M. S. flat (1''} \times 3/8'') &= 72 \times 3 \times 1.28 = 276.5 \text{ lbs.} \\ \text{M. S. channels (4''} \times 2'') &= 36 \times 7.09 \times 2 = 510.5 \text{ lbs.} \\ (5) \text{ Weight of main beams (22''} \times 7'') &= 5 \times 36 \times 75 = 13500 \text{ lbs.} \\ (6) \text{ Weight of cross beam (2''} \times 5'') &= 6 \times 13 \times 32 = 2496 \text{ lbs.} \\ &= 32758 \text{ lbs.}\end{aligned}$$

Allowing for revetting etc. total load on end lift beam = 35,000 lbs.

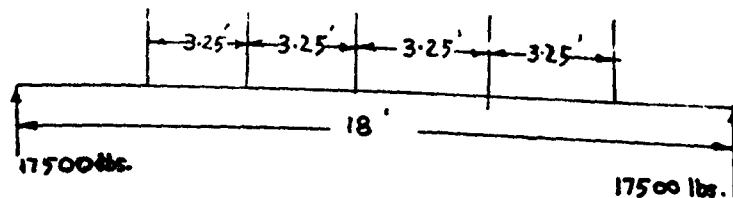


Fig. 8.

$$\begin{aligned}
 \text{Max B. M.} &= 17,500 \times 9 - 7000 (6.5 + 3.25) \\
 &= 1,57,500 - 7000 \times 9.75 \\
 &= 1,57,500 - 68,250 \\
 &= 89,250 \text{ ft. lbs.} \\
 &= 10,71,000 \text{ in lbs.}
 \end{aligned}$$

$$Z = \frac{M}{f_s} = \frac{1071000}{8 \times 2240} = 59.77$$

Two channels $12'' \times 3\frac{1}{2}''$ will give $Z = 75.96''$ in and may be adapted for end lift beam.

$$\text{Weight of lift beam} = 4 \times 18 \times 26.37 = 1899 \text{ lbs.}$$

$$\begin{aligned}
 \therefore \text{Total weight of Decking} &= 32758 + 1899 \\
 &= 34657 \text{ lbs.} = 15.47 \text{ tons.}
 \end{aligned}$$

$$\text{Allowing for reveting etc.} = \text{Weight} = 16 \text{ tons.}$$

Design of Counter Weight:

$$\text{Total weight of Decking} = 16T$$

Weight of counter weights to be 5% less than the weight of decking.

$$\begin{aligned}
 \therefore \text{Weight of each counter weight} &= \frac{16 \times .95}{2} = 7.6 \text{ tons.} \\
 &= 17,024 \text{ lbs.}
 \end{aligned}$$

$$\begin{aligned}
 \text{The size of counter weight} &= 18' 0'' \times 3' 6'' \times 1' 0'' \\
 &= 63 \text{ cuft.}
 \end{aligned}$$

$$\text{Let volume of cast iron} = x \text{ cuft.}$$

$$\text{Volume of Concrete} = (63 - x) \text{ Cuft.}$$

$$450x - 140(63 - x) = 17024$$

$$450x - 140 \times 63 + 140x = 17024$$

$$310x = 8320$$

$$x = 26.47 \text{ Cuft.}$$

$$\text{Assuming height of C.I. Block} = 2'$$

$$\begin{aligned}
 \text{Width of C. I. Block} &= \frac{26.47}{18 \times 2} = 0.7353' \\
 &= 8.82'' \text{ or } 9''
 \end{aligned}$$

DESIGN OF REINFORCEMENT

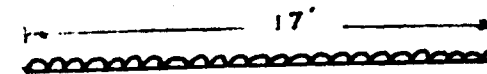


Fig. 9.

$$\begin{aligned}
 \text{B. M.} &= \frac{WL}{8} = \frac{7.6 \times 17}{8} \times 12 \times 2340 \\
 &= 4,34,000 \text{ in lbs.}
 \end{aligned}$$

Area of Steel reinforcement

$$= \frac{4,34,000}{18000 \times 36} = 0.67 \text{ sq. in.}$$

Therefore 4 Nos. $\frac{1}{2}''$ dia. M. S. Rods at the Top and bottom are adapted

$$= 0.196 \times 4$$

$$= 0.784 \text{ sq. in.}$$

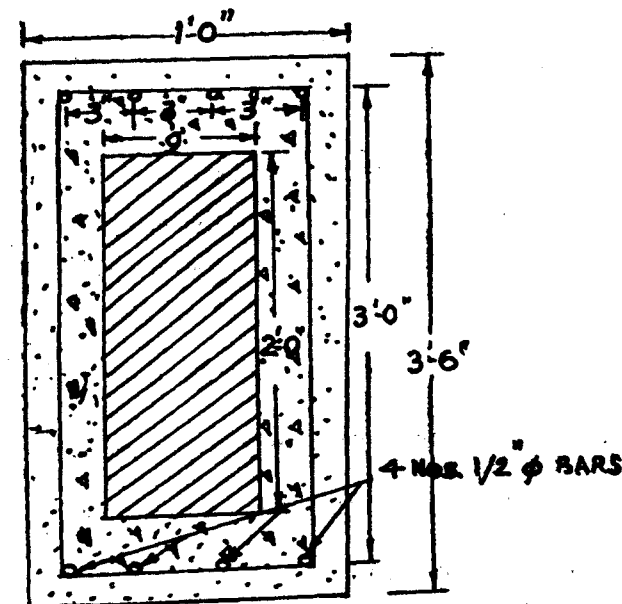


Fig. 10.

LIFT SPAN ACROSS KADUVAYAR

Lifting Mechanism—Design Calculation:

Weight of the lift span: 13 tons.

The weight of the counter weights should be about 5% less than the weight to be balanced for stability of the deck (Waddell).

Then the weight of the two counter weights: $16 \times .95 = 15.2$ tons.

Power required for raising and lowering vertical lift span is in general approximately 1 H.P. for every 15 or 20 tons of moving loads. This load includes the weight of the lift span, counter weights and all moving parts.

Assuming total weight = $16 \times 1 = 32$ tons.

Approximate H. P. required $\frac{32}{15} = 2.13$ H.P.

Detailed Calculation for Power:

1. Force required to overcome all unbalanced loads (Including unbalanced chain) assumed as 5% of the weight of the lift span: i.e. $16 \times 2240 \times 0.05 = 1792$ lbs.

2. Friction between guides and tracks taken as 15% of the normal wind load on the vertical projection.

Area exposed to wind:

Main beam $36' \times 22'/12$	=	66 sq. ft.
Kerb and flooring $36' \times 1/12$	=	51 sq. ft.
Hand railings $36' \times 1/12 \times 2.5$	=	7.5 sq. ft.
Top rail $36' \times 2/12$	=	6 sq. ft.
		130.5 sq. ft.

For ordinary operations a wind pressure of 2 lbs. per sq. ft. may be assumed. But the site of this bridge being at sea side and assuming that the lift span is to be operated in emergencies under a hurricane wind pressure of 30 lbs./sq. ft.

Wind load = $30 \times 130.5 = 3915$ lbs.

Therefore Friction = $3915 \times 0.15 = 587.25$ lbs.

3. Force required to accelerate the lift span.

$$F = \frac{W \times a}{g} \text{ where } F = \text{Force required in lbs.}$$

a Acceleration in ft./sec.

W Total weight of all moving loads in lbs.

g Acceleration due to gravity — 32.2.

Assuming total time for lighting 90 seconds.

Time for acceleration 20 "

Time for retardation 15 "

Then time for uniform motion 55 "

Let uniform speed = x ft./sec.

Assuming average speed during acceleration = $\frac{x}{2}$ = ft./sec.

and average speed during retardation $\times 1/2$ ft./sec.

$$20 \times 1/2 + 55 + 15 \times 1/2 = 14.33.$$

$$\frac{145}{2} \times = 14.33.$$

$$\times = 0.1976 \text{ ft./sec. say } 11.9 \text{ ft./minute.}$$

$$\text{Acceleration} = \frac{0.1976}{20} = \text{say } 0.01 \text{ ft./sec.}^2.$$

$$\text{Therefore Force required for acceleration: } \frac{32 \times 2240 \times 0.01}{32.2} = 22.25 \text{ lbs.}$$

$$\therefore \text{Total maximum force at the sheave rim} = 1792 + 587.25 + 22.25 = 2401.5 \text{ lbs.}$$

$$\text{Torque} = 2401.5 \times 30.5/2 = 36,640 \text{ in lbs.} = 3053 \text{ ft. lbs.}$$

$$\therefore \text{H. P. required at the sheaves} = \frac{2N(\text{Torque})}{33000} \quad N = \text{R. P. M. of sheave.}$$

$$= \frac{11.9 \times 12}{\times 30.5}$$

i.e. 11.9 minute.

$$= \frac{2 \times 11.9 \times 12 \times 3053}{\times 30.5 \times 33000} = 0.8862 \text{ H. P.}$$

Assuming efficiency of worm gear — 50% and efficiency of bevel gear 75%

$$\text{Motor H. P. } 0.8862 \times 100/50 \times 100/75 = 2.34 \text{ H. P.}$$

Therefore a 3 H. P. Motor may be used.

DESIGN SHAFTS.

I. Main Shafts:

R. P. M. of Motor = 750.

Reduction through chain drive and work gear = 150: 1.

∴ R. P. M. of main shaft $\frac{750}{150} = 5$ R. P. M.

H. P. of the motor = 3 H. P.

Efficiency of worm gear 50%.

Therefore H. P. transmitted by main shaft $= 3 \times \frac{50}{100} \times \frac{1}{2} = .75$ H. P.

Assuming diameter of shaft = 2.5 material mild steel.

Twisting moment in the shaft $= \frac{H. P. \times 33000}{2 \pi N}$

$$\frac{.75 \times 33000}{2 \pi \times 5} = 788 \text{ ft. lbs.} = 9455 \text{ in lbs.}$$

Allowing key way factor = 1.2

Twisting moment $M = 9455 \times 1.2 = 11,347 \text{ in lbs.}$

Unsupported length of the shaft

$$l = 50 \sqrt[3]{\frac{M}{d^2}} = 50 \sqrt[3]{\frac{11,347}{2.5^2}} = 92.10 \text{ inches} = 7.38 \text{ ft.}$$

∴ Bearings can be assumed at 6 ft. centres.

$$\begin{aligned} \text{B. M. due to self weight of shaft} &= 2 \frac{W l}{8} \\ &= \frac{16.53 \times 6 \times 6 \times 12}{8} = 8926 \text{ in lbs.} = M_1. \end{aligned}$$

$$\text{Equivalent B. M.} = \frac{1}{2} M_1 + \frac{1}{2} \sqrt{M_1^2 + M_2^2}$$

$$+ \frac{1}{2} \times 892.8 + \frac{1}{2} \sqrt{892.8^2 + 11,347^2}$$

$$= 447 + \frac{1}{2} \sqrt{12,93,80,000}$$

$$\text{Eq. B. M. } 447 + \frac{1}{2} \times 11390 = 447 + 5695 = 6142 \text{ in lbs.}$$

$$\text{Stress due to B. M.} = \frac{M}{Z} = \frac{6142 \times 32}{\pi \times 2.5^3} = 4000 \text{ lbs./sq. in.}$$

$$\text{Considering equivalent Twisting moment } T_e = \sqrt{M_1^2 + M_2^2} = 11390$$

$$\text{Shear stress due to } T_e = \frac{11390 \times 16}{\pi \times 2.5^3} = 3712 \text{ lbs./sq. in.}$$

∴ The mild steel shafting with a safe stress of 6000 lbs./sq. in. can be used.

II. End Shafts: (a) Portion under the sheave.

$$\text{R. P. M. of end shaft} = \frac{5}{3.5} = 1.43 \text{ RPM}$$

$$\text{H. P. of the end shaft} = 0.75 \times \frac{75}{100} \times \frac{1}{2} = \frac{9}{32} = 0.281 \text{ H. P.}$$

$$\text{Twisting moment} = \frac{0.281 \times 33000}{2 \pi \times 1.43}$$

$$= 1032 \text{ in lbs.}$$

Keyway factor 1.2

$$\therefore M_1 = 1238 \text{ in lbs.}$$

Vertical load on shaft $= T_1 + T_2$

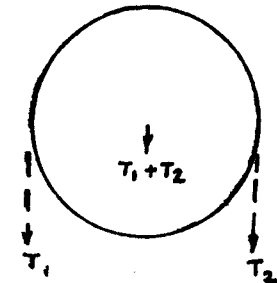


Fig. 11

$$T_1 = \frac{\text{Weight of Deck and chains} + \text{Friction} + \text{Inertice force}}{4}$$

$$= \frac{16 \times 2240 + 960 + 597 + 22.25}{4} = \frac{37,409}{4} = 9,352 \text{ lbs.}$$

$$T_2 = \frac{\text{Weight of counter weights and chains}}{4}$$

$$\begin{aligned} &= \frac{15.2 \times 2240 + 80}{4} \\ &= \frac{34,048 + 80}{4} = \frac{34,128}{4} = 8,532 \text{ lbs.} \end{aligned}$$

$$\begin{aligned} W = T_1 + T_2 &= 9362 + 3532 \\ &= 17,884 \text{ lbs.} \end{aligned}$$

$$\begin{aligned} \text{Max. B. M. } \frac{WL}{4} &= \frac{17884 \times 12}{4} \\ &= 53,652 \text{ in lbs.} \end{aligned}$$

$$\text{Add for B. M. due to self weight} = 348$$

$$\underline{54,000 \text{ in lbs.}}$$

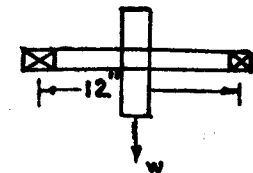


Fig. 12

$$\begin{aligned}
 \text{Eq. B. M.} &= \frac{1}{2} M_1 + \frac{1}{2} \sqrt{M_1^2 + M_2^2} \\
 &= \frac{1}{2} \times 63,652 + \frac{1}{2} \sqrt{54,002^2 + 1233^2} \\
 &= 26,826 + \frac{1}{2} = 24,140 \\
 &= 26,826 + \frac{1}{2} \times 54,140 - 26,826 + 27,070 = 53,896 \text{ in lbs.}
 \end{aligned}$$

Assuming 3" dia shaft.

$$\text{Max. stress (fiber)} = \frac{53,896 \times 32}{\pi \times 3^3} = 20,330 \text{ lbs. sq. in.}$$

$$\text{Eq. T Moment } \sqrt{M_1^2 + M_2^2} = 53,660 \text{ in lbs.}$$

$$\text{Max. shear stress} = \frac{53,896 \times 16}{\pi \times 3^3} = 10,160 \text{ lbs.}$$

Therefore N1 alloy steel with ultimate tensile strength of 100 tons sq. in. should be used for the portion of the shaft under the sheave.

$$\text{Factory safety} = \frac{100 \times 2240}{2033} = 11.62$$

$$\text{For portion outside the sheave B. M. due to self weight} = \frac{WL}{8}$$

$$\begin{aligned}
 l &= 50^3 \sqrt{d^3} = 30^3 \sqrt{3^3} \\
 &= 50 \times 2.0801 = 104.0' = 8.7 \text{ ft.}
 \end{aligned}$$

Actual $l = 8 \text{ ft.}$... safe.

Portion outside the sheave.

$$\text{Self weight} = W = 23.8 \text{ lbs./ft.}$$

$$\text{B. M.} = \frac{23.8 \times 8 \times 8 \times 12}{8} = 2285 \text{ in lbs.}$$

$$\begin{aligned}
 \text{Sq. B. M.} &= \frac{1}{2} M_1 + \frac{1}{2} \sqrt{M_1^2 + M_2^2} \\
 &= \frac{1}{2} \times 2285 \times \frac{1}{2} + \frac{1}{2} \sqrt{2285^2 + 1238^2} \\
 &= 1143 + \frac{1}{2} - 2599 = 1145 + 2599 \\
 &= 1143 + 1300 = 2443 \text{ in lbs.}
 \end{aligned}$$

$$\text{Max. bending stress} = \frac{M}{2} = \frac{2443 \times 32}{\pi \times 3^3} = 921.5 \text{ lbs./sq. in.}$$

Therefore this is safe for mild steel.

Therefore the portion outside of the sheave may be M. S.

Hand Operating Mechanism:

In the case of failure of electric supply manual operation will be employed.

A suitable platform and ladder for the men to go to the top of the structure will be provided.

Torque required at sheave: 3053 ft. lbs.

Gear ratio from sheave to works: $3.5 \times 50 = 175$.

Efficiency of reduction = $.5 \times .75 = .375$.

Therefore Torque required at hand wheel $\frac{3053}{175 \times .375} = 46.52 \text{ ft. lbs.}$

Assuming that a men will exert a force of 35 to 40 lbs. (say 35 lbs.) on a lever with a speed of 150 to 160 ft. per min. (say 150 ft. per min.) Hool and Kinne page 177) Radius of hand wheel $\frac{46.5}{35} = 1.32 \text{ ft.}$

Therefore A hand wheel of 1' 6" radius will be provided.

Time required for lifting by one men = $\frac{14.33 \times 3.5 \times 50}{150} = 16.7 \text{ minutes.}$

$$\text{R. P. M. of hand wheel} = \frac{150}{\pi \times 1.32} = 157 \text{ RPM}$$

A suitable platform and windless will be provided for hand operation (Hool and Kinne pg. 177).

The Driving Mechanism:

The main drive is from a reversible type AC slippering Induction Motor of 3 H.P. and 750 R.P.M.

The reduction worm gear with chain drive reduces this speed by 150 times (worm gear ratio 50: 1 chain drive 3: 1).

Thus the speed of the main shaft will be 5 R.P.M.

The drive from the main shaft to the end shaft through the pinion and level gear units at each end reduces this speed in the ratio 35: 10.

Thus the speed of the end shafts is $\frac{5}{3.5} = 1.43 \text{ R.P.M.}$

Therefore the 30.5" diam. sheaves will revolve at a speed of 1.43 R.P.M. which will give a peripheral speed of about 11.5 feet per minute.

Thus the total lifting height of 14' 4" will be covered in approximately 90 seconds.

COLOGNE HIGHWAY BRIDGE

This continuous plate-girder bridge, to be built over the Rhine, will have a cable-braced center span of 990 ft. and a cable-braced end span of 494 ft. Since this bridge is still in the planning and design stage, and since the City of Cologne reserves the right to first publication of details, only general comments follow. Erection is scheduled to start in the spring of 1958, and presumably the bridge will be completed by the summer of 1959.

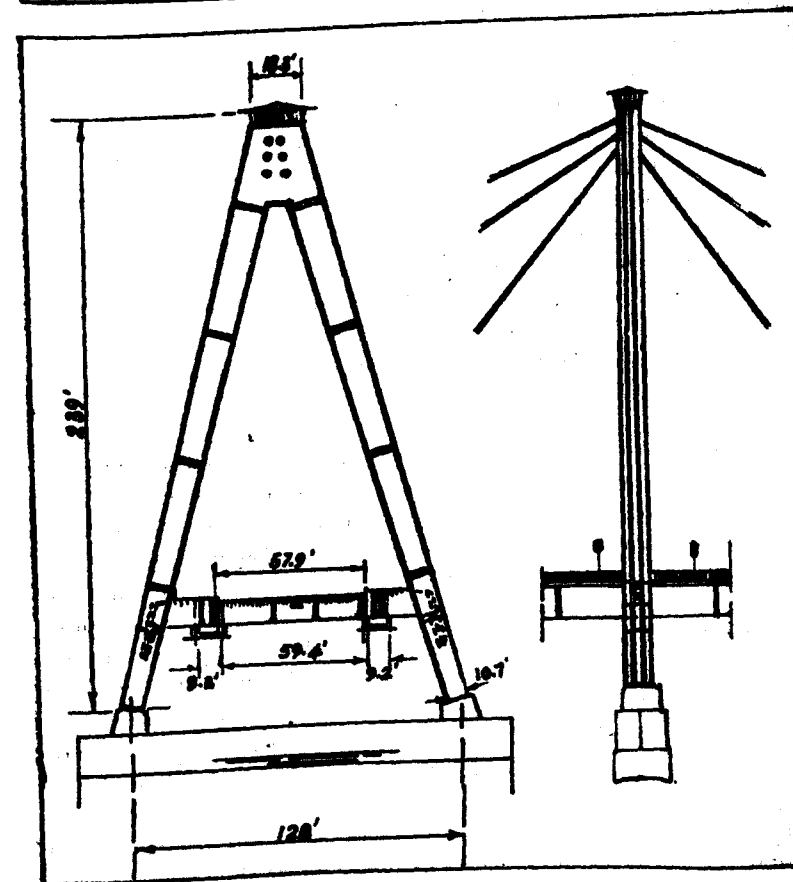
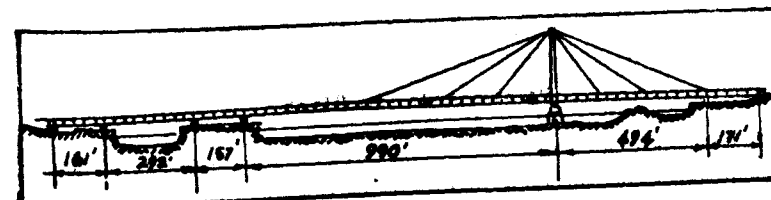
Interestingly, this bridge will have only one pylon on the east side of the Rhine. (The Author was informed, while in Cologne, that local towns-people revere their famous Gothic cathedral (Dom) so much that it was required that all river passengers must have an unobstructed view of the beautiful Dom).

The bridge will be chiefly of welded construction. Field joints of the main girders will be riveted, while the deck plate (believed to be orthotrope design) will be field welded. The weight will average 84 psf.

The superstructure is estimated to cost about 16 million DM (3.8 million). Design of the bridge will incorporate the latest techniques, with principal consideration given to reduction in steel weight.

Gutehoffnungshuette Sterkrade AG Oberhausen—Sterkrade will head up this project.

—“Engineering News-Record”,
November 7, 1957.



COLOGNE HIGHWAY CROSSING WILL BE A CABLE-BRACED, CONTINUOUS PLATE GIRDER BRIDGE WITH CABLES EXTENDING FROM ONLY ONE TOWER, AN A-SHAPED PYLON.

ASSOCIATION
NEWS

The 1958 Annual Session of the Association of Madras State Highway Engineers will be held on 21st and 22nd June, 1958.

The members are requested to fill in the form appended below and send the same to the Secretary at an early date.

Dear Sir,

(i) I shall be attending the Annual Session ☐

(ii) I wish to join in the following :

Lunch on 21—6—58

@ Rs. 2/- ☐ VEG. ☐ NON-VEG. ☐

Subscription dinner on 21—6—58

@ Rs. 6/- ☐ VEG. ☐ NON-VEG. ☐

Excursion—Transport & Tea on 22—6—58

@ Rs. 2/- ☐

(iii) I enclose Rs.....by P. O. ☐

M. O. ☐

Cheque ☐

Yours faithfully

Member,

ASSOCIATION OF MADRAS STATE
HIGHWAY ENGINEERS.

Roll No.....

Note.—Please fill in the gaps and tick ☐ where applicable.

124062

ROADS AND TOURISTS

"The first thing that must strike any commonsense motorist in South India to-day is the appalling waste of public money incurred in modernising our roads, treating them with concrete and tarmac for the benefit of the fast and heavy traffic of our current civilisation. I am not saying this work should not have been done—of course it should have been done. Still less am I saying it has not been well done—most of it has been excellently done. Where the wastage comes in is in doing it and then annulling it—or allowing it to be annulled. What, please, is the use of spending uncounted crores in order to make these roads suitable for fast traffic and then allowing them to be so bunkered and blocked by pedestrians, and by livestock either unattended or attended by herds who have no intention of herding, that the safe maximum is brought down to the 30 m.p.h. for which a travelled surface would have surfaced? So that driving on them becomes a nightmare and a man calls himself lucky to have got home with the death of only a pig on his hands and not that of a child?

That he will have the death of something is a virtual certainty. One can thread one's way through a mosaic of goats, donkeys, bullocks, buffaloes for a time but in the end, sooner or later, the predestined victim makes the fatal leap. Nor are these by any means the worst: in fact, I notice that in the course of the twenty odd years since I last did much motoring in India these animals have learned a certain amount of road sense. But *Homo Sapiens* has learned none. All he has done is to increase his numbers five-fold and his obstructiveness considerably. He does things which no human driver could predict and from the consequences of which no human driver, without luck and the grace of God, could save him. I plead guilty, on these terms, to having knocked down and injured one woman in the course of some 30,000 miles—and Heaven alone knows how it was not one hundred or one thousand. And oh! the horning and homing and borning and the snatching at the brakes and the cursing and shouting on what should have been—for the money spent on it—a fine fast enjoyable piece of open road.

Now I am convinced that all this is quite unnecessary. There must be laws or by-laws in existence making it an offence to obstruct the highway, whether with your goats or with your fool self. They only need enforcing. But—

discipline and democracy again—nobody does it. A few Police cycle-patrols armed with nice whippy stinging canes a posse of village servants shoving the malcontents to the margins; a score of sharp sentences, for straying unattended livestock; in short, half a dozen orders and the energy to carry them out—these would do it. Even with the powers I had thirty years ago I would guarantee to show, on any given road, a vast improvement in six months and a complete clearance in twelve. It can be done and easily. But—democracy and discipline—everybody must do just as he likes and blow everybody else."

—"The traveller in South India", Hilton Brown,
Extracted from Hindu.

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The plan for a rocket establishment at Peenemunde was approved in 1936. Before the evening of the day on which it was authorised, the site had been purchased for 750,000 marks, or about £ 58,000."

—Extracts from "*Science Unfolds the Future*"
By J. G. Crowther.

Alas! The Highways Research Station Land Acquisition was proposed during 1951; but still it is not completed!



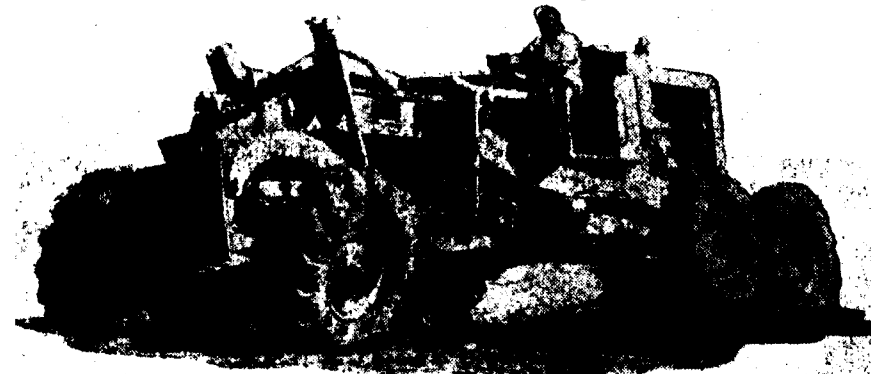
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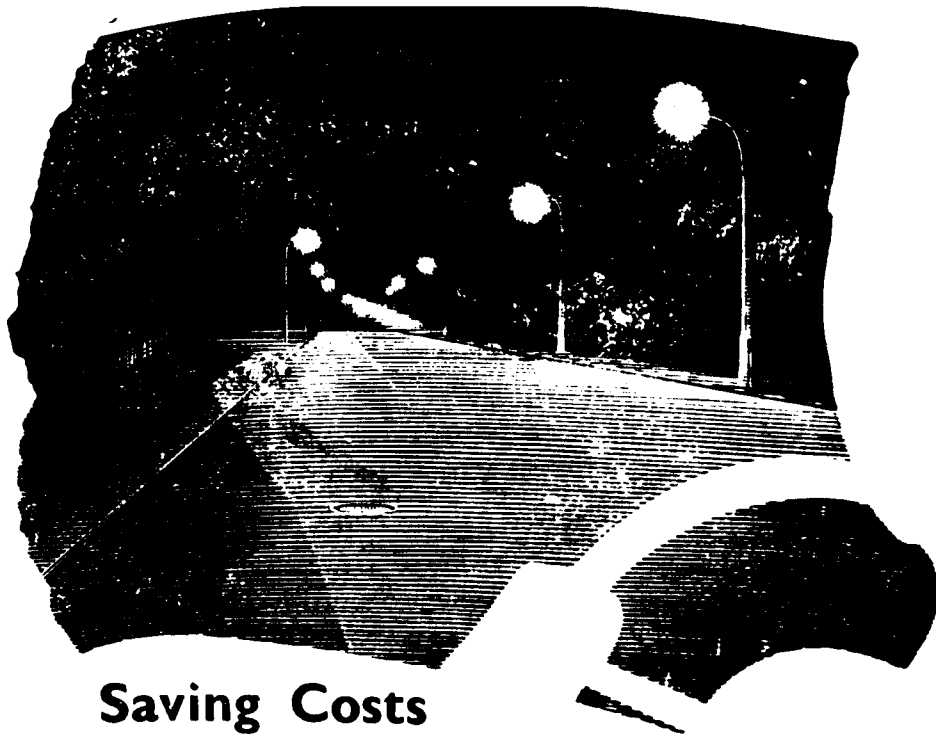
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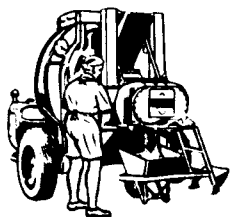
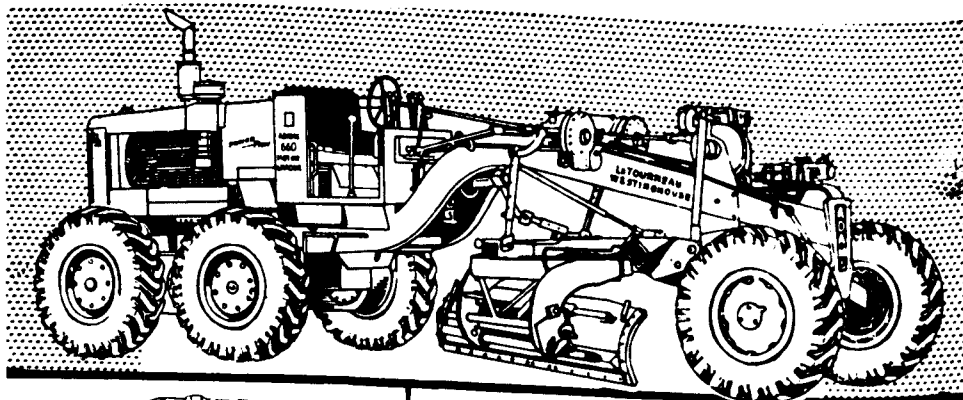


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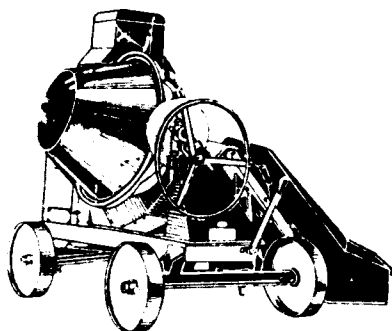
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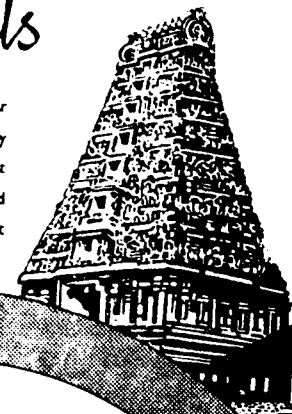
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