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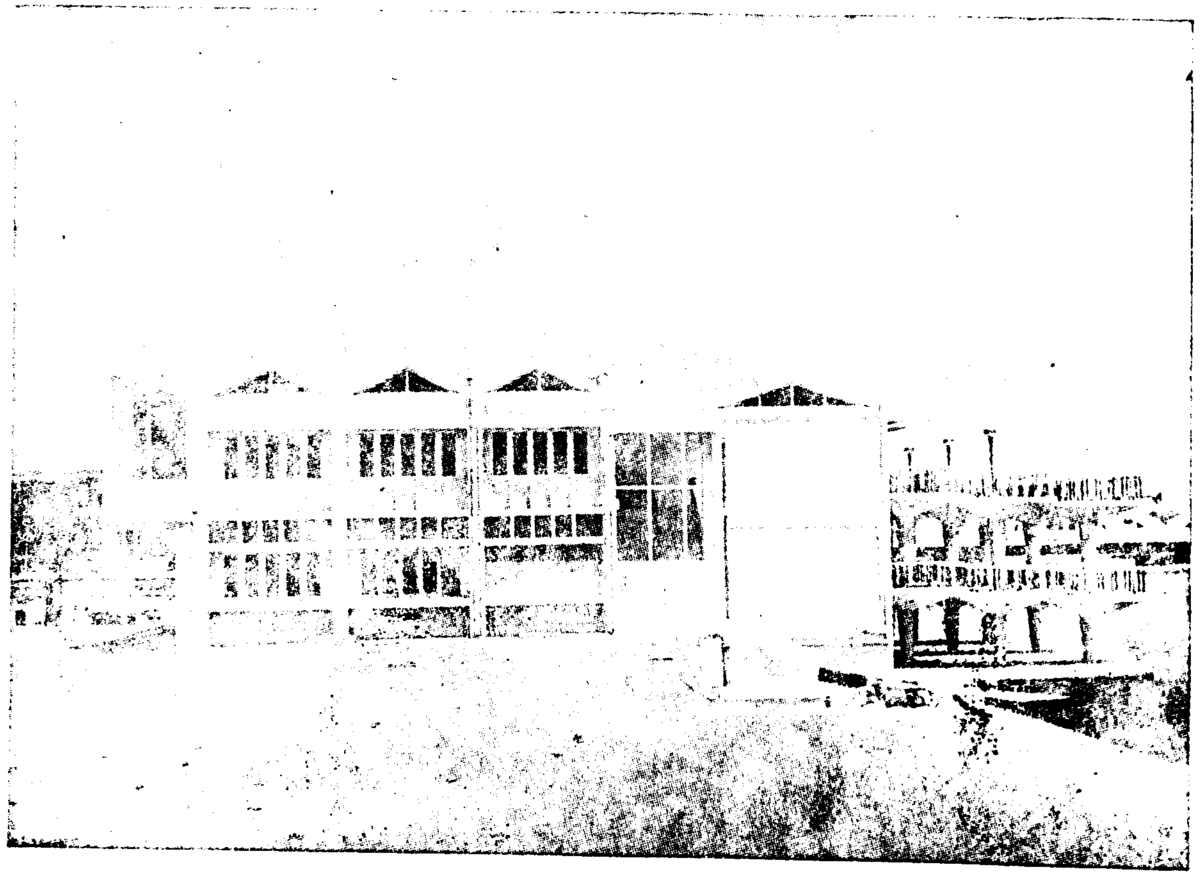
BOAT CLUB WESTERN ELEVATION

BOAT CLUB EASTERN ELEVATION





# THE NEW IRRIGATION ERA

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# DESIGN OF BRIDGE OVER SPILLWAY

BY

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** D. K. NAGARAJAN, B.E. (Hons.),
*** M. SADASIVAM, B.E.,

## 1.0. Introduction.

A spillway bridge located in a dam is required when a highway has been routed over the dam or when bulkhead gates for river outlets or spillway crest gates require the use of travelling crane for their operation. One of the several types of bridges either of steel or reinforced concrete construction may be chosen depending upon the length of span, head restrictions or requirements above the crest or purely aesthetic reasons. Where there is no highway across the dam, the spillway bridge may consist simply of two steel girders or reinforced concrete beams supporting crane rails and a walkway. Where the bridge is to be used for a highway, more care is to be given to its appearance.

Design criteria for highway bridges usally conform to the standard specifications, modified to satisfy the local regulations and any particular requirements of the project. Girders, floor beams and floor slabs are designed to support the loading classification given in the standard specification consistent to the class of traffic to use the bridge.

Design loads usually include the dead load of the structure and the live load due to traffic.

Other considerations which must be satisfied include camber, grouting of load slab, adequate storm drainage and roadway lighting.

## 2.0. Design Criteria.

### 2.1. Span of bridge and width of roadway.

The span of the girders of bridge is the spacing of piers in the spillway and the width of the roadway required in the bridge are determined in consideration with the hydraulic aspects and the other conditions at site.

### 2.2. Type of bridge.

Simply supported T beam bridges are one of the most popular of bridge types for spans ranging from 30' upto about 60 or 70 feet. It is really an economical adoption of the slab type for span where the slab thickness and consequently the dead load, become excessively heavy. For other spans of spillway bridges also, ranges between 30' to 60' generally, the T beam bridge is usually adopted.

The deck slab acts first as a roadway slab stressed transversely and secondly as a flange providing compression area in the T beam, stressed longitudinally. The floor arrangements for T beam bridges are divided into two types.

(1) Girders and the slab type in which concrete slabs span between longitudinal girders.

(2) Girder, floor beam and two way slab type in which the panels of the floor slab are supported along four edges by the floor beams and the longitudinal girders.

The arrangement of the type (2) is used to reduce the thickness of slab consistent with the minimum thickness required when designing for class AA (I.R.C) loading which will otherwise require thick slabs.

### 2.3. Girder and slab type of floor.

In this group are included bridges consisting of longitudinal main girders with concrete slabs spanning between girders, cross beams, when used do not carry any load directly but only serve to stiffen the structure effectively. The number of main girders in the cross section of bridge may be reduced without increasing their spacing by cantilevering the roadway slab on both sides. The length of the cantilever should be selected so as to equalise the loadings on all girders but it should not be so long as to cause appreciable deflections of the cantilevers under heavy moving loads.

### 2.4. Spacing of Main Girders.

Greater spacing of the longitudinal main girders affects to a large extent the cost of the bridge, therefore comparative estimates of several arrangements of girders should be made before the final arrangement is adopted. Close spacing of girders

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means thinner slab and a large number of main girders, wide spacing of girders on the other hand, means thicker slabs but a smaller number of girders. The economical spacing of main girders is the spacing which gives the least possible sum total of the cost of the formworks, including its vertical supports and bracing and the cost of materials in the slab and the girders.

Comparing two designs with different spacing of main girders shows that closer spacing of girders results in the saving in the cost of materials but at the same time, incurs additional cost of formwork.

Relative economy of the two arrangements depends upon which of the two values are larger and this in turn, depends upon the relation between the unit cost of materials andthe unit cost of formwork.

### 2.5. Cross Beams.

In the floor designs of this type cross beams are used at each end of the bridge to stiffen the edges of the deck slab which might otherwise suffer severely under the sudden application of load as vehicles come on to the bridge. Cross beams also help in stiffenlng the girders laterally.

When cross beams are intended to distribute loading laterally they should be made strong enough to perform the desired function.

When cross beams are considered only as stiffening struts, they should be provided with bottom reinforcement equal in cross section to about 0.3% of the effective cross section of the cross beams.

### 3.0 Loading to be adopted

1) The different types of loadings as per I.R.C. standards, namely AA Class, A class and B Class loadings are given in the appended sketch with full details.

2) 'B' Class loading as per M.D.S S. (vide P.310) consisting of one road roller and one lorry.

(sketch 2 appended with details)

Any one of the above loadings, suitable to the site conditions may be adopted. Generally "B" class loadings are adopted for most of the spillway bridges.

### 3.1. Design of Slabs for Floor Arrangements in First Type. The Effective Span Lengths of Slab.

The deck slab is not only continuous, but is built monolithic with the supporting guides and is also provided with fillets at its junction with the girder. In such cases, where fillets make an angle of 45° or more with the axis of the slab, the effective span shall be measured from the section where the combined depth of the slab and fillet is atleast one and half times the thickness of the slab.

### 3.2. Bending Moments in Slab.

Positive bending moments at the centre of each slab panel and negative bending moments at the edge of the supporting girders are to be evaluated as these are the critical sections. Provisions also should be made for negative bending moments in the centre of each panel which are produced when the panel under consideration is not loaded while the adjoining panels are loaded.

For deck slabs, the width will usually be more than three times the span.

Pigeands' curves are employed in finding out the maximum positive moments in the slabs. Since the results obtained are for simply supported slabs, a reduction of 20% is made in the moments to allow for the continuity in the slabs. For negative moments at the edge of the supports the same magnitude as net positive moment is assumed.

### 3.3. Depth of Slab.

The depth of slab is determined by the maximum total positive bending moment. The deck slab should not be made less than 7" thick as it has to be strong enough to support heavy wheel loads, and as it must have sufficient stiffness to transfer load from each beam to its neighbours.

### 3.4. Slab Reinforcement.

Main reinforcement of slabs consists of top and bottom bars placed in the direction of the span of the slab.

From one half to two thirds of the bottom bars may be bent up and extended into the adjoining panel to serve as negative bending moment reinforcement.

Negative bending moment reinforcement is also required in the central part of each span and the amount of this reinforcement should not be



less than one third of the bottom reinforcements in that span.

### 3.5. Bending Moment in Slab with Cantilevers.

As class "AA" tractor and bogie cannot be placed closer than 4' from the face of the kerb, only the wheel loads of class A vehicle need be considered for computing maximum bending moments in cantilever, unless "B" class load is adopted.

Dispersion of wheel loads in cantilever slab of T beam bridge may be calculated from the following formula. $e = 1.2X + W$ (1) for a single concentrated load where $e$ = effective width of slab.

$X$ = the distance of the c.g. of the concentrated load from the face of the cantilever support.

$W$ = the breadth of concentration area of the load i.e. the dimension of the tyre or truck contact area over the road surface of the slab in a direction parallel to the supporting edge of the cantilever + twice the thickness of the wearing coat or surface finish above the structural slab. (vide clause 207.22 b of I.R.C. 1958)

### Design of Main Girders.

The girder must be wide enough to accommodate all longitudinal reinforcement without crowding and preferably without using more than three layers of longitudinal bars. The load on each girder shall be the reaction of the wheel loads, assuming the flooring between girders to act as simple beam, while one row of the wheels is being directly placed over the girder.

Then the trail is adjusted for the maximum bending moment in the span, as per the criterian for maximum bending moment for rolling loads.

### Shear in Main Girders.

Part of the main reinforcement should be ben up to provide shear steel. Bars should be bent up in pairs placed symmetrically in cross section of the beam. Vertical stirrups could always be provided, in addition and as an alternative to bent bars.

### 4.0. Type of Floor Consisting of Girders, Floor Beams and Two way Slabs.

In this type the spacing of floor beams if possible is made equal to the spacing of the longitudinal girders. As a result the slab is divided into a number of square or rectangular panels supported on four sides and reinforced by two sets of bars placed at right angles to each other.

In comparison with the other type, this arrangement has the following advantages. For the same spacing of main girders, the thickness of slab is reduced because the loads are carried in two directions, which reduces appreciably the bending moments upon the slab, the dead load upon the girder is appreciably reduced, which in turn reduces the dimensions of the girders.

### 4.1. Bending Moments of Slabs.

Bending moments in slabs are calculated using Pigeand's curve in the same manner as for slabs supported on two sides only. The free bendine moments obtained are reduced by 20% to providg for the continuity of the slab.

### 4.2. Design of Cross Beams.

The most unfavourable loading for a cross beam consists of the rear axles of as many trucks as can be there accommodated, placed side by side directly above the cross beam. No transverse dispersion of these wheel loads is allowed. Apart from this, the reactions of the otherset of rear axles located on the slab panels should be added to the loading cross beam.

A model calculation is appended to the article.

### Bibliography.

1. Design calculation for Parambikulam Spillway bridge.
2. Design Manual for Lower Bhavani Project.
3. Highways Manual.

## APPENDIX A.

### Problem :

Design a spillway bridge given the following data :

Data : Width of roadway = 14' 0" clear

| | |
|---|---|
| Pier thickness | = 10' 0" |
| Length of clear span | = 42' 0" |
| Effective span | = 42' + 2' = 44' 0" |

Loading = I.R.C. Class A

Design Stresses: Mix. 1:2:4, c = 750 psi

t = 18000 psi., m = 15

R.M. = 126 $bd^2$, L.A. = 0.87 d.

Bond stress = 100 psi. Shear stress = 75 psi.

Draw to scale a dimensional drawing showing R.C. details of spillway bridge.

**Design Proper:**

Loading: IRC Class A.

Impact factor for concrete bridge, I.F. $= \frac{15}{20+L}$

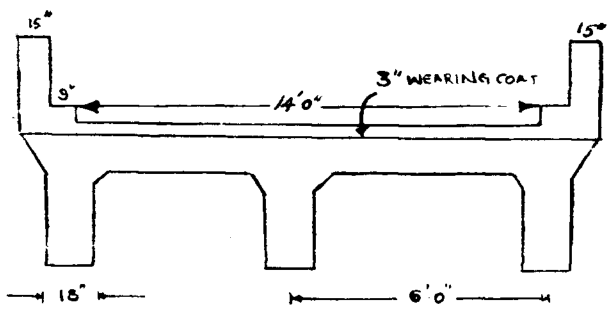
for spans between 10' and 150' and limited to 0.5' for span less than 10' (L is the effective span in feet).

## I. Design of Deck Slab:

Span of the slab = 6' 0"—1' 6" = 4' 6" or 54"

In the case (of continuous spans, the clear span is taken as effective span since fillets are provided)

### i. Dead Load:




Wearing coat 3" thick $= \frac{3}{12} \times 150 = 37.5$ lbs/sq.ft.

Self weight of slab $= \frac{7}{12} \times 150 = 87.5$ lbs/sq. ft.
(7" slab assumed)

Total 125 lbs/sq. ft.

Maximum dead load B.M. $= \frac{WL^2}{8}$

$= 125 \times 4.5 \times 4.5 \times \frac{12}{8} = 3796$ or 3800 in lbs.

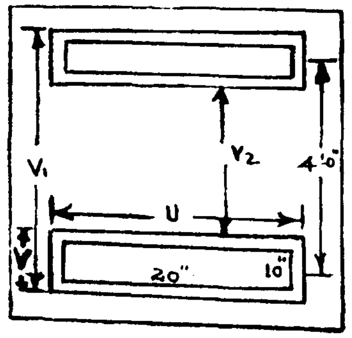
### ii Live load:

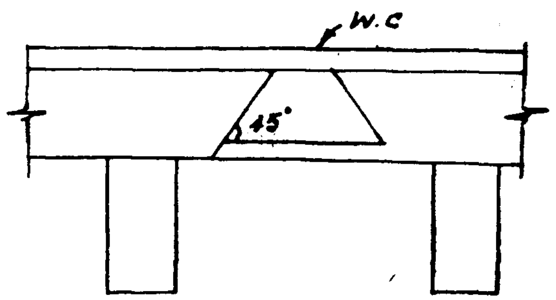
I.F. $= \frac{15}{20+L} = \frac{15}{20+4.5} = 0.612$ limited to 0.5 since the span is less than 10'

Hence the total laod has to be multiplied by 1.5.

For maximum B.M. the wheels must be placed at the centre of the span.

The dispersion of a concentrated load through the total thickness of the road formation in concrete slab is considered as acting at an angle of 45° from the edge of the contact area to the centre of the lower layer of reinforcement.






U = 20 + 2 (3 + 5.5) = 37", assuming that the centre of reinforcement is at a distance of 1.5" from the bottom of slab. V = 10 + 2(3 + 5.5) = 27."

For maximum B.M. the two adjacent loads of 12500 lbs are placed at the centre of the span, the effect of the other loads in the chain are neglected. Referring to R. C. Designer's Hand Book by Reynolds page 233 Table No. 14 D,

$K = \frac{L_L}{L_B} = \frac{44 \times 12}{54} = 9.8$ This is more than 2.5 and so it can be taken as infinity and the corresponding table shoud be referred to find the maximum B.M.

Intensity of load/sq.in $= \frac{\text{Wheel load x impact}}{\text{area of dispersion}}$

$= \frac{12{,}500 \times 1.5}{37 \times 27} = 18.77$ lbs/Sq. in

First it is imagined that the area $V_1 \times U$ is loaded with an intensity of 18.77 lbs/Sq.in and the corresponding moment is calculated. Then the moment due to the load on the area $V_2 \times U$ is calculated and deducted from the previous moment.

$L_B = 54"$ U = 37"

$V_1 = 48 + 27 = 75"$

$V_2 = 48 - 27 = 21"$

$\frac{V_1}{L_B} = \frac{75}{54} = 1.389"$; $\frac{U}{L_B} = \frac{17}{54} = 0.6852"$

For the above factor $m_1 = 0.09$, $m_2 = 9.915$.

**Transverse bending moment.**

Total load ($m_1$ + 0.15 $m_2$)

$= 18.77 \times 75 \times 37\ (0.09 + 0.15 \times 0.015) \times 12 = 57650$ in lbs.

**Longitudinal Bending Moment:**

$= 18.77 \times 75 \times 37\ (0.015 + 0.09 \times 0.15) \times 12 = 17810$ in lbs.

Considering the load on area $V_2$ x U,

$\frac{V_2}{L_B} = \frac{21}{54} = 0.3889"$ and $\frac{U}{L_B} = \frac{37}{54} = 0.6852"$

For the above factor $m_1 = 0.12$, $m_2 = 0.075$

Transverse B.M $= 18.77 \times 21 \times 37 (0.12 + 0.15 + 0.075) \times 12$
$= 22980$ in lbs.

Longitudinal B.M $= 18.77 \times 21 \times 37 (0.075 + 0.15 \times 0.12) \times 12$
$= 16{,}280$ in lbs.

**Net-transverse B.M.**

due to live load $= 57650 - 22980 = 34670$ in lbs.

2

**Net longitudinal B. M.**

= 17810 − 16280 = 1530 in lbs.

Transverse B.M. due to live load = 34670 in lbs.

do do dead load = 3800 in lbs.

Total moment. 38470 in lbs.

The depth of slab required $= \sqrt{\frac{38470}{126 \times 12}} = 5.04"$

Total thickness of 7" is adopted. Allowing 1" clear cover and adopting ½" dia. rods the effective depth = 7 − (100 + 0.25) = 5.75"

Area of steel $= \frac{38470}{0.87 \times 5.75 \times 18000} = 0.4274$ Sq.",

provide ½" $\phi$ at 5.5" c/c

Negative B.M. at the interior support $= \frac{WL^2}{8}$ (as per M.D S.S.) = 38470 in lbs.

Negative B.M. at the support

$= \frac{WL^2}{16} = \frac{38470}{2} = 19235$ in lbs.

The greater of these two values will be adopted as the distance between the maximum wheel loads is 4 ft, the effective width of these two wheels will overlap. The total effective width for the two wheels will be 8.441 ft, and so the effective width for one wheel is 4.2205 ft. Shear force per unit width of slab due to Live load at support

$= \frac{1.5 \times 12500}{4.22} \times \frac{52}{72} =$ 3209 lbs.

Dead load shear force = 0.62 × 125 × 6 = 465 lbs.

Total shear force 3674 lbs.

∴ Shear stress $= \frac{3674}{12 \times 0.87 \times 5.75} = 61.20$ psi (Safe)

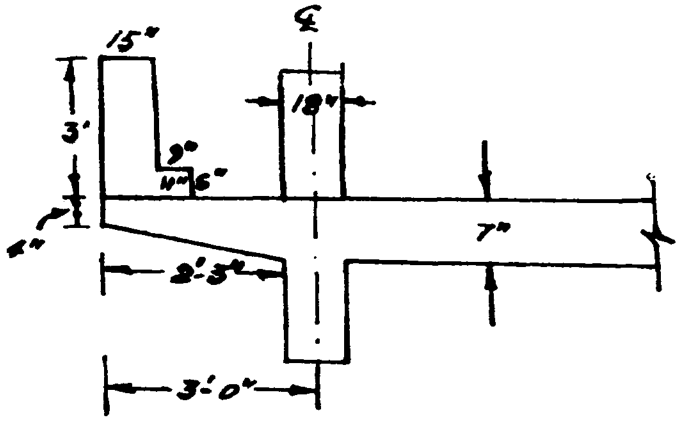
Since the design is not safe against bond stress provide main reinforcement allowing bond stress of 175 psi.

$175 = \frac{3674 \times X}{0.87 \times 5.75 \times 12 \times 1.57}$ or X = 4.488" or 4.5"

Hence adopt a spacing of 4.5"c/c for the main reinforcement in slab using 1/2" ø bars.

**Design of cantilever portion of slab:**

1) Live load :- Maximum B. M. in the cantilever will occur when the edge of the wheel is at a distance of 6" from the kerb which is the minimum distance allowed. But in this case the wheel load falls well within the rib of T beam. There will be no live load moment produced in the cantilever.



Bending Moment :

Due to self weight of slab adopting 4" thick at the end,

$= \frac{(4+7)}{2\times12}\times\frac{27}{12}\times150\times12.27 = 1898$ in lbs.

| | |
|---|---|
| Due to parapet $= \frac{15\times36}{144}\times150\times19.5 =$ | 10960 |
| Due to kerb $= \frac{9\times11}{144}\times150\times7.5 -$ | 773 |
| Wearing coat $= \frac{3\times3}{144}\times150\times3/2$ - | 14 |
| Total Moment | = 13645 |

Negative B. M. in the cantilever (13645 in lbs) is less than the B. M. in the cantinuous slab and support (19230 in lbs). Hence alternate bars of the slab main reinfercement can be cranked to take negative B. M. i.e , ½" ø @ 9" c/c.

### Distributers for the slab :

$\frac{\text{Area of main steel}}{\sqrt{L}} = \frac{0.4274}{\sqrt{4.5}} = 0.2015$ Sq. in

∴ Adopt 3/8" ø @ 6" c/c

Same distributing bars are provided in the cantilever also.

### Design of T. Beam for Road Bridge :

One end of the beam is prevented from long movement and the other end is kept free to allow freedom of movement. The T beam is designed considering it as simply supported.

Clear span = 42' 0"

Effective span = clearspan + centre to centre of bed blocks

= 42' + 2' = 44'0"

Spacing of beams = 6' 0"

Effective width of flange is the least of

1) 1/3 Span = ⅓×44×12 = 176"

2) 12 ds + br = 12×7 + 18 = 102"

3) Centre to centre of beams = 72"

Therefore the effective width of flange = 72" which is the least of the three.

Dead Loads.

Assume the depth of rib to be 1/12 of span i.e. $\frac{1}{12}\times44\times12 = 44$"

Weight of T beam/ft. run

$= \frac{(72\times7)+(18\times37)\times150}{144} = 1219$ lbs.

Weight of wearing coat/ft. run

$= \frac{72\times3\times150}{144} = 225$ lbs

Total dead load per ft. run 1444 lbs/ft.

Maximum B.M. $= \frac{1444\times44\times44}{8\times1000} = 349.45$ ft. Kips

Maximum dead load shear $= \frac{1444\times44}{2\times1000} = 31.77$ Kips

*Live load* : Since the road is only 14' wide only one standard vehicle can pass. To produce the maximum effect on the beam one set of wheels of the vehicles should travel along the centre line of the beam. The effect of the load from the other wheel will not be felt on this beam since the beams are spaced at 6'0" centres and the distance between the wheel centres is 6' 0".

Load on T beam = One wheel load × (impact factor + 1)

= P × (impact factor + 1)

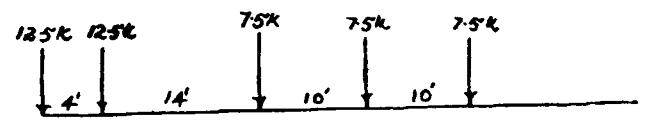
Impact facter $= \frac{15}{20+L} = \frac{15}{20+44}$

= 0.2344, the wheel loads travelling on T beam are increased by 1.2344.

### Maximum B. M. due to live load :

Maximum load that could be accommodated is the following :





The C.G. of the above load system for the first 12.5 Kips load

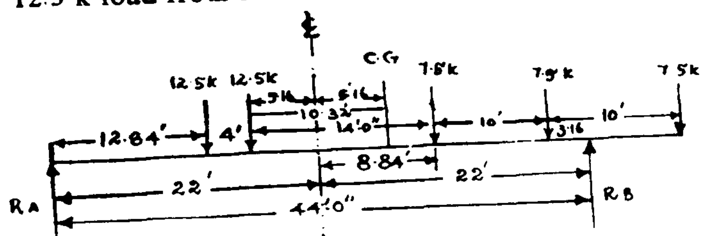
$= \frac{1}{47.5}(4\times12.5+18\times7.5+28\times7.5+38\times7.5)$

= 14.32 ft.

To get the maximum B.M. we have to arrange the load system according to theorem, when a number of loads move across the span, the maximum B.M. will occur under the heavier load when that load and the centre of gravity of the system are equidistant from the centre line of the span.

### Case I

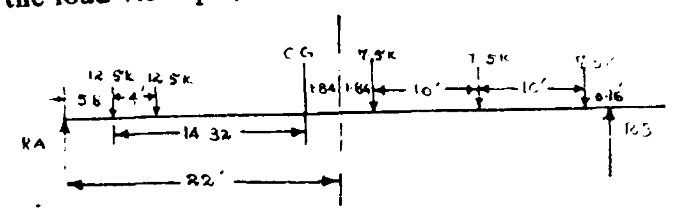
Assuming the maximum B.M. is under the 2nd 12.5 k load from left.



As the load 7.5 falls outside the span the loads must be readjusted.

### Case II

Assuming the maximum B. M. occurs under the load 7.5 kips (1st from left).

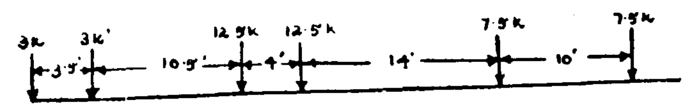


RB×44 = 47.5×20.16 ∴ R.B = 21.76 and RA = 25.74

Taking moment about the point of application of load, 7.5 K nearst the centre line.

B.M. = 21.76×20.16 — 7.5×20 — 7.5×10 = 213.7 ft. Kips.

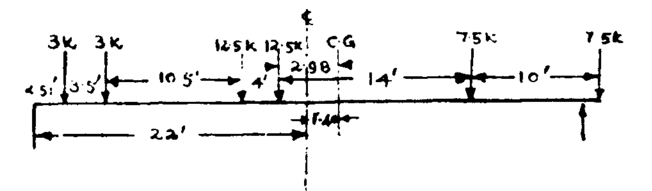
### Case III



Taking the first six loads, the C.G of the system from the first load

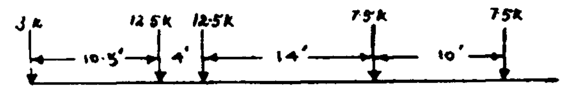
$= \frac{3\times3.5+12.5\times14+12.5\times18+7.5\times32+7.5\times42}{46}$

= 20.98'



Since the last load falls outside the span the loads may be readjusted.

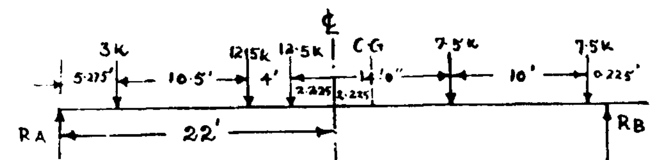
### Case IV



To find the C.G. from the first load i.e. 3 k load,

$\frac{12.5\times10.5+12.5\times14.5+7.5\times28.5+7.5\times38.5}{43} = 18.95$

Arranging the load on the span as shown below:



RB × 44 = 43 × 24.225; RB = 23.68 and RA = 19.32

B.M. = 19.32×19.775−3×14.5−12.5×4 = 288.7ft.Kips

Allowing 1.2344 for impact factor,

B. M. = 288.7 × 1.2344 = 356.98 ft. kips.

Dead load B.M. = 349.45 ft. Kips.

Total B.M. = 706.43 ft. kips or 707 ft. kips

### Economical depth of T beam:

$d = \frac{t}{2}+\sqrt{\frac{My}{fs.b}}$, fs = 18000 psi, b = 18" t = 7"

$r = \frac{\text{cost of steel/cft}}{\text{cost of concrete/cft}} = 60$ and M = 707×12 in Kips

$d = \frac{7}{2}+\sqrt{\frac{707\times12\times1000\times60}{18\times18000}} = 43.13$ or 43"

Size of rib = 18"×36"

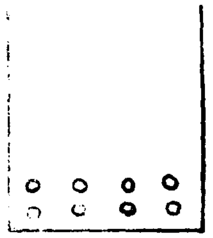
Effective depth = 43 - (240.5 + 1.5 + 0.75) = 38.25"

Area of steel required = $\dfrac{707\times12\times1000}{0.87\times38.25\times18000}$ = 1416 sq. in.

4 rods of 1½φ for bottom row and 4 rods of 1½" for top row will give a total area of 14.14 Sq. in.

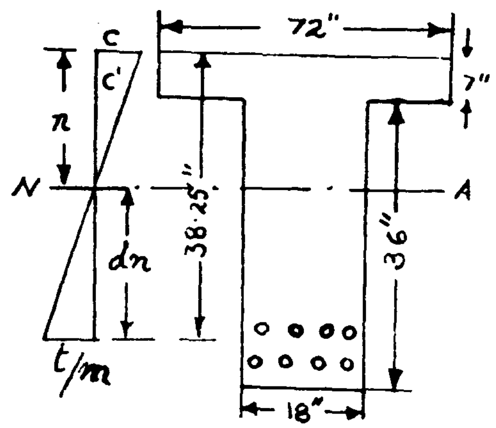
**ACCOMMODATION OF RODS**

| | | |
|---|---|---|
| 4 rods of 1½" | φ = | 6" |
| 2 stirrups of 1/2" φ | = | 1" |
| 3 int. spacing 1½" | = | 4.5" |
| 2 end covers 2" | = | 4" |
| | | 15.5" |



Hence 1[illegible]" width is sufficient.

**ACTUAL EFFECTIVE DFPTH PROVIDED :**




Total depth = 43"

| | | | |
|---|---|---|---|
| **DEDUCT FOR** | Bottom cover | = | 2" |
| | diameter of stirrup | = | 0.5" |
| | Bottom row | = | 1.5" |
| | Spare rod | = | 0.75" |
| | | | 4.75" |

effective depth = 43"—4.75" = 38.25"

**Check for stresses :**

Taking moments about N. A.,

$$B.\ ds\left(n-\frac{ds}{2}\right) = m\ At\ (d-n)$$

$$72\times7\ (n-3.5) = 15\times14.14\ (38.25-n)$$

$\therefore$ n = 13.8, $C' = \dfrac{C}{3.8}\times6.8 = 0.493\ C$

depth of C. G. of Compression flange from top of flange.

$$\frac{C+2C'}{C+C'}\times\frac{ds}{3} = \frac{C+0.986\ C}{1.493\ C}\times\frac{7}{3} = 3.1"$$

Lever arm = 38.25 − 3.1 = 35.15"

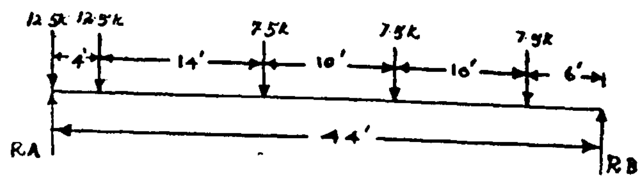
Stress carried by steel = $\dfrac{707\times12\times1000}{14.14\times35.15}$ = 17120psi

Stress in concrete = $\dfrac{17120}{15}\times\dfrac{13.8}{24.45}$ = 642 psi ∴ safe.

**DESIGN FOR SHEAR :**

**Calculation of maximum live load shear :**

For maximum shear the loads must be arranged as shown in sketch.




$$RA = \frac{12.5\times44+12.5\times40+7.5(26+16+6)}{44} = 32.05\ k$$

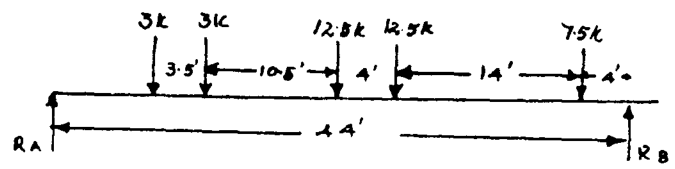
Considering the effect of impact Maximum live load shear = 32.05×1.2334 = 39.54 kips

Maximum dead load shear = 31.77 kips

Total maximum shear = 7131 Kips. This is the maximum shear force at support.

Shear stress = $\dfrac{71.31\times1000}{18\times35.15}$ = 112.7 psi

**For Maximum shear force at centre of span :**



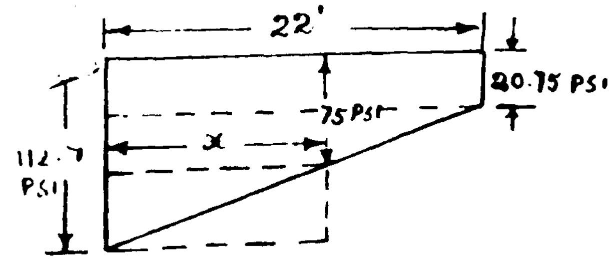

$$RA = \frac{7.5\times4+12.5\times18+12.5\times22+3(32.5+36)}{44}$$

= 16.72 kips

Shear at centre = 16.72—(3+3) = 10.72 kips considering the effect of impact, the maximum shear force at the centre = 10.72×1.2344 = 13.23Kips.



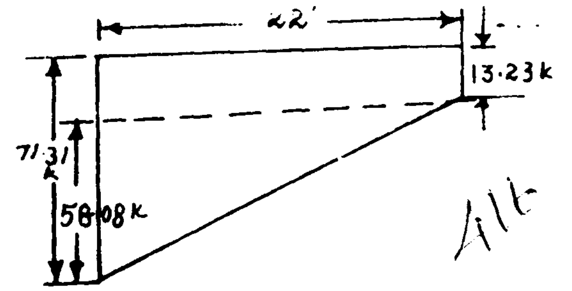
um Shear stress = $\dfrac{13.23\times1000}{18\times35.42}$ = 20.75 psi




To find the point where the shear stress is 75 psi.,

$\dfrac{x}{22} = \dfrac{37.7}{91.95}$ or x = 9.018'

**Bending up of bars to take shear force**
**Shear force diagram:**




Shear force at leverarm (2.93) from support = $\dfrac{58.08}{22}\times18.07$ = 50.30 Kips

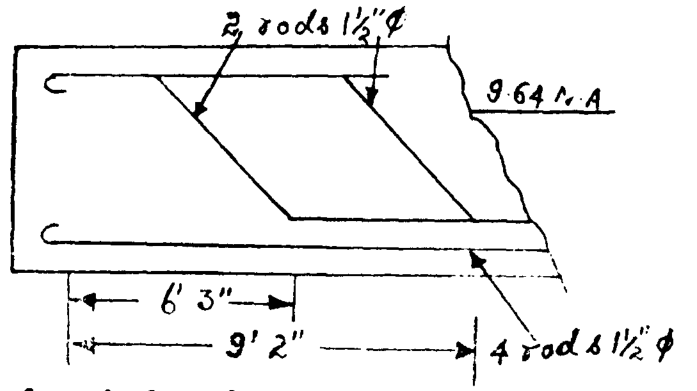
Total = 50.30 + 13.23 = 63.53 Kips

Shear force at 2 leverarm (5.86) from support = $\dfrac{58.08}{22}\times16.14$ = 42.7 Kips

Total = 42.7 + 13.23 = 55.93 Kips. Shear force for 75 psi = 35.15×18×75 = 47.82 kips. Shear taken by the two cranked bars of 1½"φ = 2×1.7671×18000×0.7071 = 44.99. kips. This is effective for a distance of one leverarm (35.15) on either side of the point where the neutral axis and cranked bar cut. Fixing up the cutting point at an distance of 36" from the face of the support, the balance shear to be resisted by the stirrups beyond the centre of support = 71.31—44.99 = 26.32 kips. Spacing ½" φ double legged stirrups = $\dfrac{2\times0.196\times18000\times36}{26320}$ = 9.651 or say 9" c/c



Another two bars of the top layer reinforement are cranked up at a distance of two leverarm i.e. 7030




from the face of the crank to the point where the cranked bar and the N.A. cross each other.

Distance from centre line = 70.84 + 12 + 26.44 = 9' 2"

**DESIGN OF R.C. BED BLOCK :**

The bed block will be continuous under all the beams and is designed as a beam with ribs of T beams as supports. The loads transmitted by all the beams are assumed to be uniformly distributed over the entire block.

Maximum load from one beam = 71.37 kips

For design purpose it is assumed that the other two beams also tramsmit the same load. Length of the bed block = 14' 0"

Assume the width of the bed block = 2'

U.D.L./ft. width = $\dfrac{71.31\text{x}3\text{x}1000}{14}$ = 15290 lbs.

Maximum + ve B.M. at middle = $\dfrac{15290\text{x}6^2\text{x}12}{10}$ = 660300 in lbs

Maximum—ve ,, = $\dfrac{15290\text{x}1^2\text{x}12}{2}$ = 91700 in lbs.

The bed block is designed for the greater B M. using 1 : 2 : 4 concrete mix.

Depth = $\sqrt{\dfrac{660300}{126\text{x}24}}$ = 14.78"

Provide over all depth = 18"

Assume ⅞" φ bars. effective depth = 18 − (1.5 + 0.5 + 0.87) = 15.625

Area of steel = $\frac{660300}{0.87 \times 15.625 \times 18000}$ = 2.698 Sq. in.

Provide 7 bars $\frac{3}{4}$ $\phi$ throughout the 2½' width of slab. To avoid inconvenience in construction equal number of bars i.e. 7 nos. 3/4" $\phi$ are provided to take up the negative B. M.

**SHEAR STRESSES:**

Considering a foot width of bed block

Maximum shear = 15290x6x0.62 = 56850 (0.62 being the coefficient from Roynolds Table No. ·)

Shear stress = $\frac{56850}{24 \times 0.87 \times 15.625}$ = 174.3 psi ∴ Provide shear reinforcement.

The maximum shear that the concrete can take without shear reinforcement = 75x0.87 x 15.625x24 = 24470 lbs. of this occurs at x ft. from support

x = $\frac{56850-24470}{56850}$ x 3 = 1.70ft.

Balance shear to be taken up by stirrup = 56850—24470 = 32380 lbs.

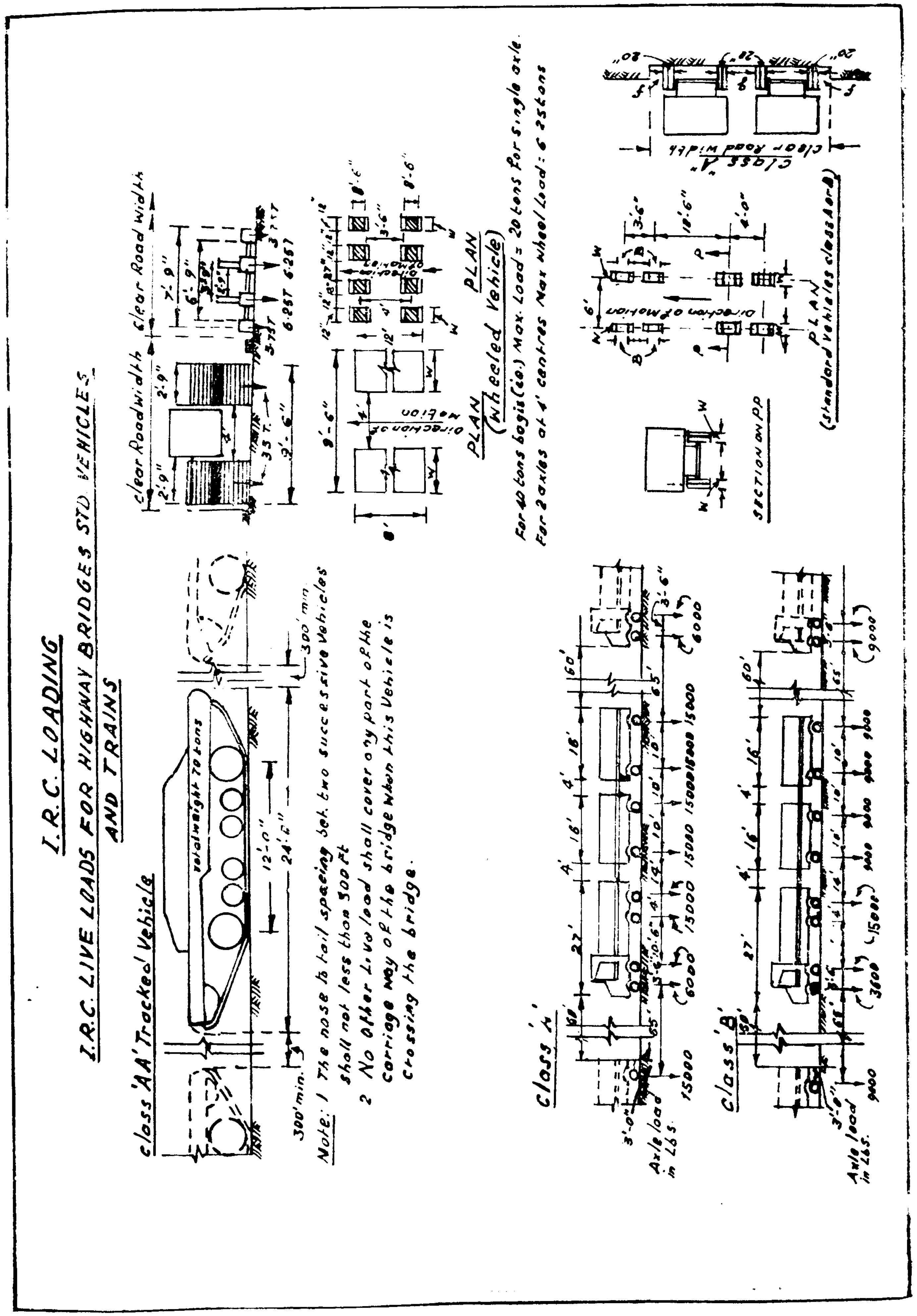
Spacing of ½" bars = $\frac{0.196 \times 2 \times 18000 \times 0.87 \times 15.625}{32380}$ = 2.9 or 2.75" c/c

Then the nominal stirrups are provided in the middle portion of supports by 12" spacing as shown in the enclosed drawing.

Check the stress induced in the concrete padding size 10½" x 16"

Total reaction at support = $\frac{71.31 \times 1000}{10.5 \times 16}$ = 424.5 psi Safe.

Detailed dimensional drawing is appended.



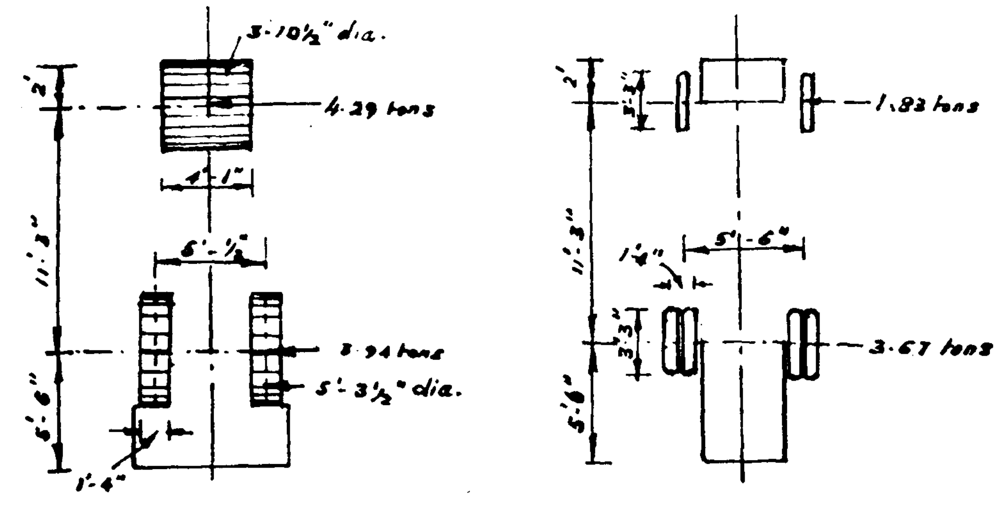
Limiting Positions of the standard Vehicle or a Train of Vehicles with reference to a Traffic Lane

**CLASS 'AA'**

| Clear Road Width | Min Value of the clearance between the Roadway face of the kerb and the outer edge of the wheel of the Track. |
|---|---|
| SINGLE LANE BRIDGE. | |
| 10' or Less<br>10' to 12'-6" | 0<br>Increasing Uniformly from 0 to 6" |
| Above 12'-6" | 1'-0" |
| MULTIPLE LANE BRIDGE | |
| Less than 18' | 2'-0" |
| 18' or Above | 4'-0" |

| Clear Road Width | g<br>Min. clear space between the adjacent edges of Wheels of passing or crossing Vehicles on a multi Lane bridge to be assumed for purposes of design. | f<br>Min. clearance between the Roadway face of the kerb and Outer edge of the Wheel to be assumed for purpose of design. |
|---|---|---|
| 16'-8" or Less | 0 | 6" for all Road width on all Bridges |
| 16'-8" to 18'-0" | Increasing Uniformly from 0 to 1'-4" | |
| 18' to 24' | Increasing Uniformly from 1'-4" to 4'-0" | |
| Above 24' | 4'-0" | |

| Class of Loading | Particulars of the standard Vehicles and trains | Axle Load | Ground contact Area: Dimension B inches | Ground contact Area: Dimension W inches |
|---|---|---|---|---|
| Class 'AA' | The Vehicle sketched above subject to the restriction that the nose to tail distance between two successive Vehicles is not Less than 300 feet. No other Live Load shall cover any part of the carriage way of the bridge when this train of Vehicle is crossing the bridge (a) Wheeled Vehicles | 20 tons | 6 | 12 |
| | (b) Tracked Vehicles | (Max) | 144 | 33 |
| Class 'A' | The train sketched above subject to the restriction that the min. space (nose to Tail) between two successive trains shall be 60'-0". No other Live Load shall cover any part of the carriage way when this train of vehicles (or trains of Vehicles in the case of Multilane bridges) is crossing the bridge | 25,000 Lbs.<br>15,000 Lbs.<br>6,000 Lbs. | 10<br>8<br>6 | 20<br>15<br>8 |
| Class 'B' | The Train sketched above subject to the restriction that the min space (nose to tail) between two successive trains shall be 60'-0". No other Live load shall cover any part of the carriage way when this train of Vehicles (or trains of vehicles in the case of a MultiLane bridge) is crossing the bridge. | 15,000 Lbs.<br>9,000 Lbs.<br>3,600 Lbs. | 8<br>6<br>5 | 16<br>12<br>7 |




Road Roller

(13·17 tons, when in full working trim)

Lorry

(11 tons, when fully loaded)

**LOADS** :- Unless otherwise specified designs shall be submitted in accordance with B class Loading - (i.e) providing for (i) one road roller (vide sketch above) 4·29 tons on front wheel, width 4'-1"; 3·94 tons on each rear wheel, width of each wheel 1'-4" and gauge length between front and rear axles 11'-3" - together with (ii) One Lorry (vide sketch above) 1·83 tons on each front wheel, 3·67 T. on each rear wheel, width of each rear wheel treated as 1'-4" and gauge length between the front and rear axles 11'-3" - together with (iii) 80 Lbs. per square foot on the remaining portion of the span. (No crowd Load shall be assumed within 5'-6" of rear axle nor 2'-0" from front axle for either roller or Lorry, nor in the width of the bridge when the roller and Lorry are in Line, in the width of the bridge) - together with (iv) 30% impact for all the wheel Loads, both for the steel-floor system and for the trusses.

# TECHNIQUE OF FOUNDATION CONSTRUCTION IN BLACK COTTON SOIL

BY
Thiru G. CHANDRAHASAN*

## SYNOPSIS

This paper first deals with the causes for swelling and shrinking of black cotton soil and their effects on structures constructed on such a soil. The existing practices of foundation construction in black cotton soil and their drawbacks are discussed. Various other constructional methods that will properly control the harmful effects of black cotton soil on structures are also outlined. In conclusion a case study of buildings constructed in black cotton soil undertaken for study by our Mechanics and Research station is discussed.

## 1.0.0. INTRODUCTION

Buildings built on expansive soil commonly known as black cotton soil in India are subjected to severe cracks. This problem has been engaging the attention of engineers and suitable methods of construction of foundation to avoid such cracks in buildings are being attempted. Proper investigation into the causes of such damages to buildings and recommendations as to how structures can be designed and constructed so as to avoid or minimise these damages will be most welcome.

## 2.0.0. CAUSES OF CRACKS

2.1. Various Research Institutions which have undertaken this problem for study have reported that cracks in buildings founded on black cotton soil are caused by the swelling and shrinking of the soil, resulting from variation in its moisture content. The soil swells when it is wet in rainy season and shrinks when it is dry during the summer months. This property of the black cotton soil is due to the presence of certain clay minerals in it. When dry, the soil is very stiff and exhibits a high bearing capacity; but when it is wet, it becomes soft with low bearing capacity. Such variations in the nature of the soil exist only upto a certain depth. Soil layers at about 3.5 m below ground level are found to be unaffected by the seasonal moisture variation.

2.2.0 It has been observed that heaving up of building constructed on black cotton soil, is not uniform as there is no uniformity in the heaving up of the ground under the building The ground in the central portion of the building appears to heave more than that at the periphery, and this results in distortion of the structure. This phenomenon can be explained only by the tendency of the moisture to accumulate in the ground at the central portion of the building. Two theories have been put forward to explain the accumulation of the moisture at the centre of the building.

2.2.1. The moisture content in the soil will flow from the region of higher temperature to the region of lower temperature. As the shady effect of the building sets up such a thermal gradient between the soil at the centre of the building and that outside, the moisture migrates to the centre of the building.

2.2.2. The construction of a building results in the elimination of forces due to evaporation and transpiration, and the inflow of moisture from outside creates a new state of moisture equilibrium totally different from that which existed in the soil before the construction of the building, namely, accumulation of more moisture at the centre of the building.

2.3.0 It is obsered from many such cases of the buildings with severe cracks, that cracks start developing only during summer months. To what extent swelling of the soil is responsible for distortions of structures constructed on black cotton soil is a matter which needs study.

2.4.0 Some engineers suggest that when a building is to be constructed on expansive soil, pre-wetting of the site can eliminate or reduce the danger of cracking of buildings. The amount of heave which a building could be subjected to, can be considerably reduced, if not eliminated by pre-wetting the site to a depth of 6m or down to the

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bottom of expansive soil layer, whichever is less. This method tends to eliminate differential movement of soil under different parts of the building, which otherwise is likely to occur owing to different rates of moisture movement in different regions of the soil under the building. But still, there is a possible desiccating influence around the building if sufficient precautions are not taken. Trees planted near the structure will disturb the moisture equilibrium and give rise to severe ground movement.

2.5.0 Many are of the opinion that development of cracks in buildings constructed on black cotton soil are due to the differential settlement caused by the uneven shrinkage of foundation soil, as development of cracks are noticed mostly during the summer months. Again the differential moisture equilibrium set up by the desiccation of the soil around the building during the summer months is said to be responsible for the differential shrinkage of the soil. When the sub soil shrinks in the summer months it is more at the ends than in the middle of the building as drying starts from the ends and also the sub soil outside may shrink and pull out the soil under the ends. As the soil under the ends of the building move out, there will be greater settlement at the ends than at the middle, and this causes tension in the superstructure in vertical and inclined cracks in the building. In the rainy season, when the sub soil under the ends of the building is wetted, it swells back to regain its original volume, thus lifting up the building if the load on the soil is within the swelling pressure of the soil. During this process, the cracks developed during the summer months tend to close but not completely because the amount of swelling of the end soil during the rainy season is likely to be less than the amount of shrinkage it has undergone during the earlier summer season, which phenomenon is explained by the fact that the shrinkage of the soil is helped by the load of the building, whereas, its swelling is resisted by the same load moreover, when once a cracks is formed, crushed particles of the masonry get in to it and prevent the complete closure of the cracks. Also when the end soil swells, it slips out, and does not lift the building very much. Thus, it is clear that when cracks are once developed, there is no possibility of complete closure of these cracks.

**2.6.0 Whether it is the swelling or shrinking of the soil that causes cracks in buildings founded on black cotton soil, it is the vertical ground** movements caused by the fluctuations in the moisture content of the soil that are responsible for such damages. The proper approach to the problem should be in the direction of stabilising the soil moisture under and around the building.

## 3.0.0 Preventive Measures

3.1.0 The following are some of the methods that are adopted for foundation construction in black cotton soil with the aim of preventing cracks in buildings.

3.1.1. Adopting 5. 57 to 8. 50 tonnes per sq.m as the safe bearing capacity for black cotton soil as the soil is of poor bearing capacity in wet condition.

3.1.2. Taking the foundation down to the soil underneath the black cotton soil and resting it on some firm soil.

3.1.3. Providing crack-control measures in the masonry such as provision of R. C. bands at different levels in the walls. Usually the foundation, plinth and lintel levels are taken for the purpose.

3.1.4. Adopting suitable design load on the soil so that the loading intensity balances the swelling pressure of the soil.

3.2.0 The above methods of construction are either costly, impracticable or do not successfully arrest the ground movement and control the damages to buildings.

3.3.0 Adopting a suitable foundation pressure so as to balance the swelling pressure of the soil may not be always possible as it is difficult to transmit foundation pressures of 16.5 to 22 tonnes per sq m through normal footings except through piers or piles, and it may lead to footing failures in some cases as the soil loses much of its strength when it becomes moist. Provision of R. C. bands has not solved the problem. As already indicated the correct approach to the problem will be to aim at ways and means of controlling the differential ground movement by maintaining a balanced soil moisture condition under and around the building or to anchor the building at a depth where the ground movement of the soil due to seasonal variation is negligible, in as economical a manner as possible.

3.4.0 When the depth of the black cotton soil, is shallow, it might be possible to carry the foundation to the inert soil below the black cotton soil,



stratum by strip footings, thus avoiding the harmful effects of the black cotton soil on the foundation. But, if the black cotton soil extends to a greater depth, it will not be possible to take the footing foundation to the inert soil. In such cases, the foundation may be taken to a reasonable depth to conform to other standards of foundation depth. A layer of sand well consolidated may be provided under the foundation for a depth of 0.3 0.6 m depending upon the nature of the foundation soil with sufficient offset on each side of the width of foundation concrete. Sand piling can also be done at convenientintervals under the strip footing. The sand bed thus provided will be in a somewhat saturated condition during the rainy season, and will occupy lesser space than it will do in the summer months, and adjusts itself as the soil below swells in moist conditions. In the summer season the clay soil under the foundation will not be in a completely dry state, and so also the sand layer above. The sand layer in moist conditions occupies more space and bulks itself when the soil below shrinks during summer. Thus, the sand layer provided under the foundation adopts itself to the swelling and shrinking of the soil below and prevents the distortion of the structure above. The sand bed provided also reduces the pressure on the clay soil as the pressure intensity due to the building is maximum close to the footing, and the sand layer takes the maximum pressure as indicated in Figure 1. The sand piling suggested (not shown in the figure) will drain the sub-soil and prevent it from getting inside the building and causing differential moisture conditions under the foundations. Thus, the differential movement of the soil will also be minimised.

3.5.0. Construction of impervious pavement 1.8 m wide alround the building at ground level is also recommended. This method helps in reducing the drying of the soil close to the building and maintaining the moisture equilibrium under and around the building. Another method recommended is provision of 1 8 m wide flexible water proof apron alround at a depth of 0 6 m below the level as shown in Figure 2. The apron will consist of 15 cm thick lime concrete which should extent a little into the wall by means of a groove cut in the masonry, so as to prevent the evaporation of the moisture fron the joint between the apron and the building. The proper time for providing the apron is towards the end of monsoon when the soil is saturated.

3.6.0 The provision of sand filling under the foundation and pavement alround the building is usually recommended by this Research Division (Soil Mechanics and Research Division, P.W.D. Madras) and has been successfully adopted in various places in Tamil Nadu.

3.7.0 A well maintained garden is also recommended in certain cases. This will assist in establishing equilibrium of moisture movement from and towards the building. But planting of big trees near the structure should be completely avoided as the roots of trees absorb moisture from the soil alround and desiccate it. If trees are to be planted near the building their distance should be at least equal to their height as the lateral extension of root is more or less the same as the height. Constructional operation such as cutting for roads, drainage etc.. close to a structure should be avoided as this will result in reduction of soil moisture with the consequent shrinkage of the soil beneath the foundations of the structrue. Foundation construction close to a water course should also be avoided as the rise and fall of water level in the stream will effect the soil moisture equilibrium under the foundation In buildings constructed close to water courses cracks are noticed in many places.

3.8 0 Sometimes, ponding the area under the floor to an optimum condition before or after the foundations have been constructed is practised. The soil is allowed to get wet and swell before the fill is placed In such a case, it is necessary to see that the soil does not swell excessively and soften to such an extent that reconsolidation will lead to its settlement under the weight of the floor and the fill.

3.9.0 Excavation for foundations should not also be allowed to become excessively dry. The dried and shrunken clay will, subsequent to construction, swell again by absorption of moisture. This can result in heaving of floors and other structural damages. On the other hand, undertaking construction during the dry season and completing it before the onset of monsoon has proved successful.

3.1.0 As already stated, the vertical ground movement, which is the root cause of cracks in building is negligible at a depth of about 3 5m, and as such a good solution to avoid the bad effects of black cotton soil on a structure will be to take the foundation to this depth in as economical a way as possible. If the foundations are to be taken to this depth by conventional strip footings, it will involve deep cutting and will prove uneconomical.

A raft would also be expensive needing heavy reinforcements. Therefore, a modification of this method in the form of under-reamed piles (Fig. 3) has been found to be most suitable for such soil. It consists of 3.6 m long bored piles 20 to 30 cm in diameter at 1.5 m to 3.0 m centres with bottom belled out to 2¼ times the stem diameter. A. R. C. grade beam rests on the pile with a clearance of 8 cm above the ground. The construction of foundation involves the following steps.

3 10.1. The design of piles and beams is first prepared. Boring guides are then fixed at proper positions and bore holes of the required size are made using a manually operated spiral earth auger. The bases of the bore holes are enlarged by an under-reamed tool. The reinforcement is then introduced and concrete poured in the bore hole. The plinth beam, which is kept clear of the ground by 8 cm is then provided to support the superstructure. This type of foundation has been found to be 30 percent cheaper than the traditional strip footings and 50 percent cheaper than the raft foundation.

## 4.00 A case study of building constructed in black Cotton soil :

4.1.0 In conclusion a study of buildings constructed in black cotton soil undertaken for study by our Soil Mechanics and Research Station is discussed. The study relates to a series of buildings constructed at Avaniapuram near Madurai for Weavers Co-operative Society Colony. The foundation of the buildings had been subjected to alternate drying and wetting as it had not been taken to sufficient depth below ground level and almost all the buildings have developed cracks. The buildings were inspected and suitable remedial measures were suggested.

4.2.0 There are 150 houses in the colony and a plan of one unit consisting of two houses and rough sketches showing the cracks developed in various walls of the building are annexed for refererence (See Fig No. 4). The building provides a living room, a work room, and a kitchen and a verandah with attached sanitary annexes in an open yard in the rear. Of these rooms, the living room and the verandha are terraced and the rest tiled with mangalore tiles. The foundation has been taken only upto 2' below ground level and this insufficient provision of foundation depth has resulted in structural damage to the building. The walls in the superstructure are of 0' 9" thick and those below basement upto foundation concrete are of 1' 3" thick. The basement is about 1' above ground level, The walls in the superstructure are of brick masonry in lime mortar and the basement walls are of R. R. masonry. The foundation soil is highly clayey. A report of test of a sample of the soil taken form the area is also annexed for reference.

4.3.0 As the foundation is at a very shallow depth, it had been severally affected by alternate drying and wetting. In dry condition the shrinkage of the soil had caused settlement of the outer walls. Local enquiry also revealed that the cracks were developing only in summar. The outer walls with load at top had tilted away from the building as they settled as the drying starts from the outher edge of the foundation and developed cracks all over the walls and the tilting was perciptible. (Vide Cracks marked $C_1$).

4.4.0 In almost all the building the tilting of walls have caused cracks adjacent to the bottom of windows. This is due to the fact that only the portions of the masonry adjacent to the windows are load bearing and so settle more as the soil below shrinks. As such they tilt more than the portions of the masonry below the windows which are not load bearing and break away developing cracks.

4.5.0 Because of the tilting of outside wall, cracks have developed in the cross walls close to their junctions with the outside wall, and also near corners of rooms. This can be seen from the cracks marked $C_2$ in the sketches enclosed.

4.6.0 The cracks marked $C_3$ in the sketches are due to the failure of masonry in the absence of lintels over openings.

4.7.0 A careful study of the cracks developed in the outside wall indicated in the sketches enclosed will reveal that the cracks developed are due to the differential loading in the walls which has resulted in the differential settlement of the walls as the foundation soil shrinks in summer. The portion of the wall AB is load bearing as it carries the R. C. roof. The portion BC is of tiled roof and this wall being a side wall of the tiled roof portion is not so much load bearing as AB and portion DE is one side wall of the open yard. It can be clearly seen that the cracks marked $C_4$ are due to the diferential settlement of the walls consequent on the differential loading on the walls.



4.8.0 The only remedy at this stage will be to prevent the alternate drying and wetting of the soil under the the outside wall by providing a pavement alround the building. This will stop further damage to the building. A 3" thick stone jelly concrete payement for 3' width alround the building has been suggested along with other recommendations made for rectifying certain defeets noticed in the building. A sketch showing the recommended pavement is enclosed for reference - see sketch 6.

4.9.0 The above study is about a case of structral damages of a building brought up by the differential moisture equilibriums set up by the desication of the soil alround the building and the effects were very severe as the foundation had not been taken to sufficient depth. If the foundation had been taken to sufficient depth the soil moisture variations and desications would not have been so much as to cause such damage to the building.

## 5.0.0 Concluding Remarks.

On a detailed study of the problems of foundation construction in black cotton soil, it has been found that it is the differential moisture condition in the soil that brings about uneven ground movement under around a building which causes damage to the building. If effective measures are taken to maintain moisture equilibrium in the foundation soil, the bad effects of such a soil can be avoided. Under-reamed pile foundations offer a satisfastory solution to the problem of construction of foundations in black cotton soil.

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A report of test of a sample of the soil taken from the area, Weavers Co-operative Society Colony buildings constructed at Avaniapuram near Madurai.

| | | |
|---|---|---|
| Sand | % | 34 |
| Silt | % | 16 |
| Clay | % | 50 |
| Liquid limit | % | 77 |
| Plasticity index | % | 63 |
| Classification | | CH (Fat clay) |
| Free Swell | % | 117 |

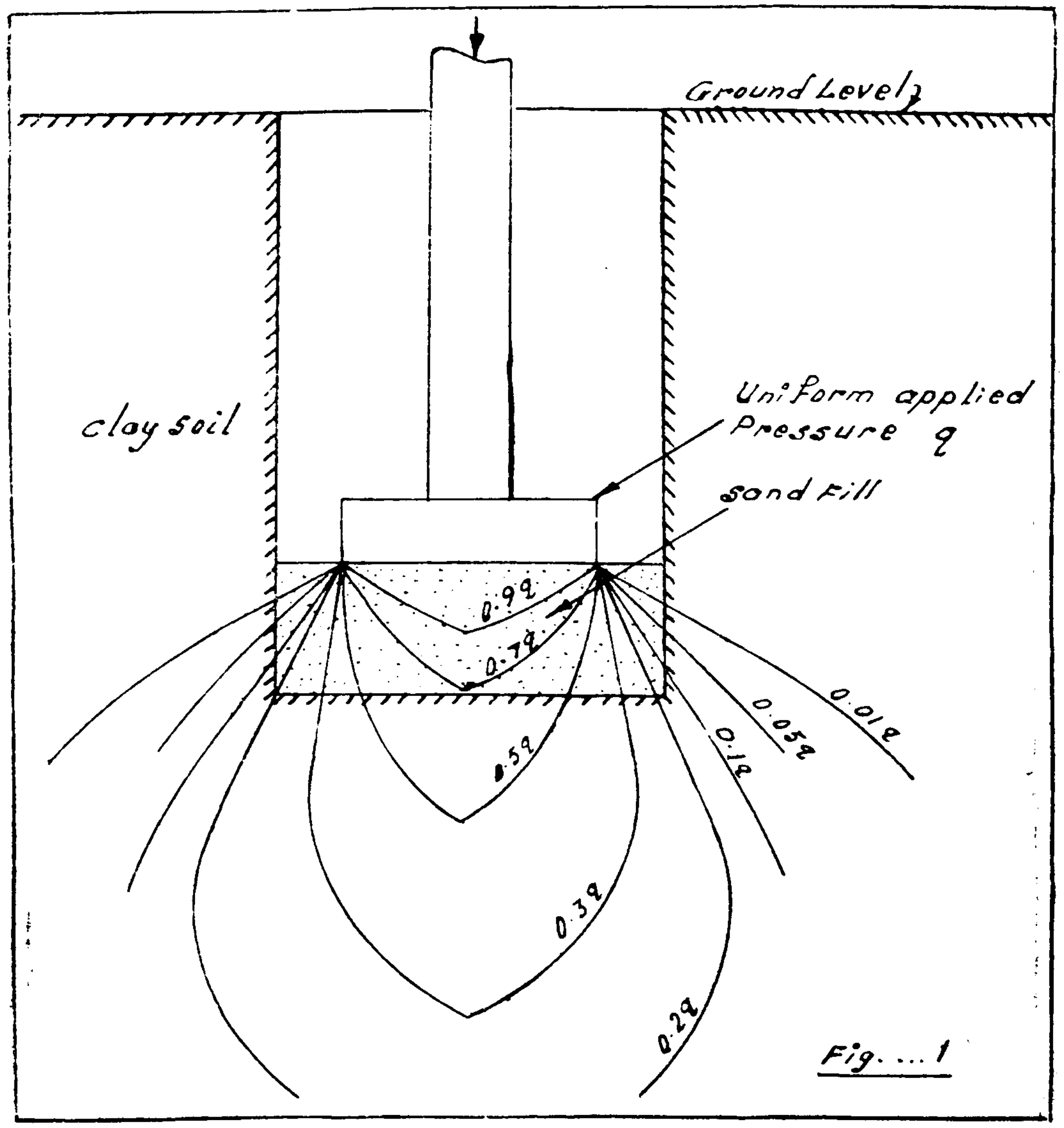
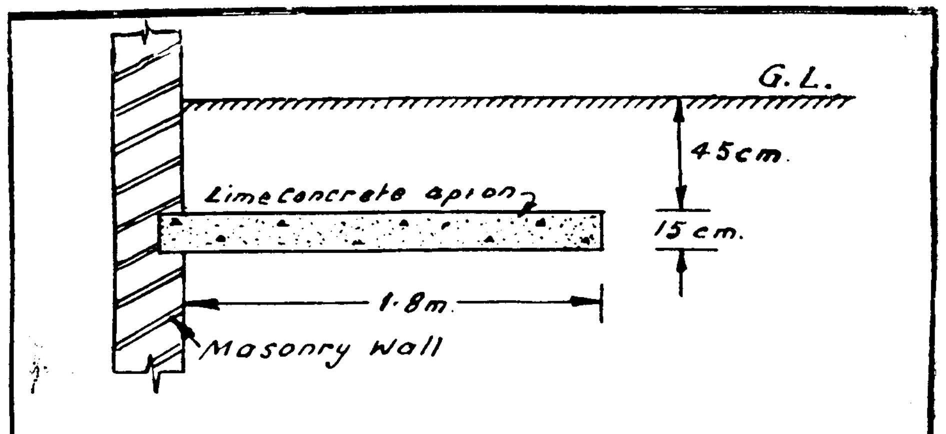


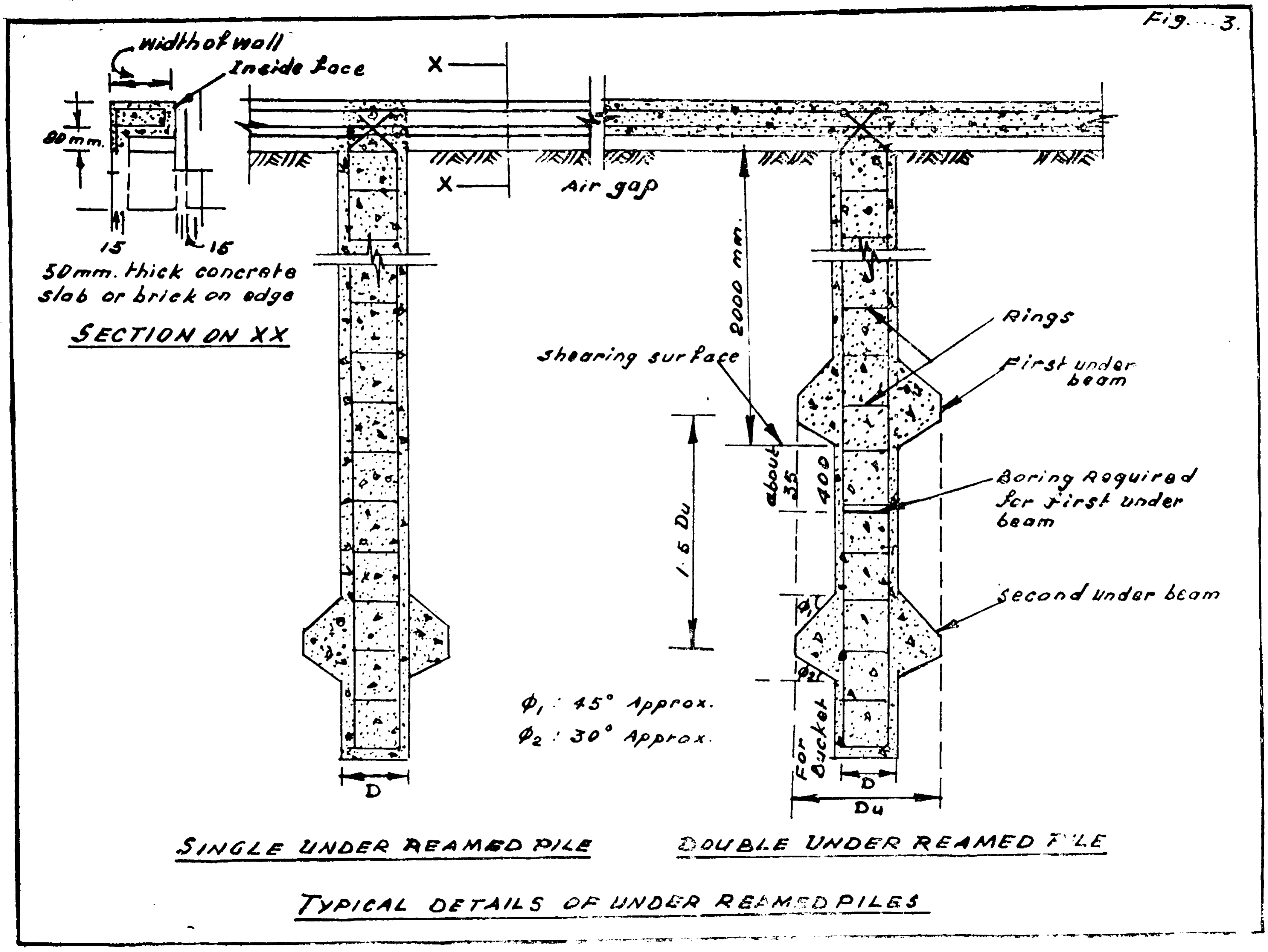

Fig....1




APRON BELOW GROUND LEVEL

Fig...2.

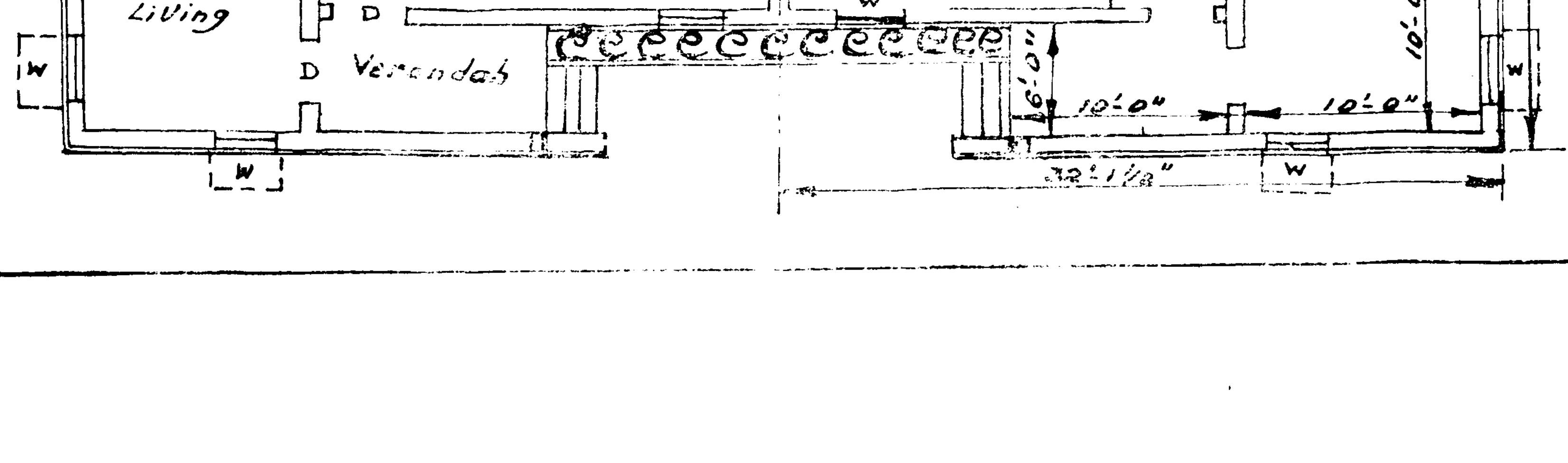
Fig. 3.



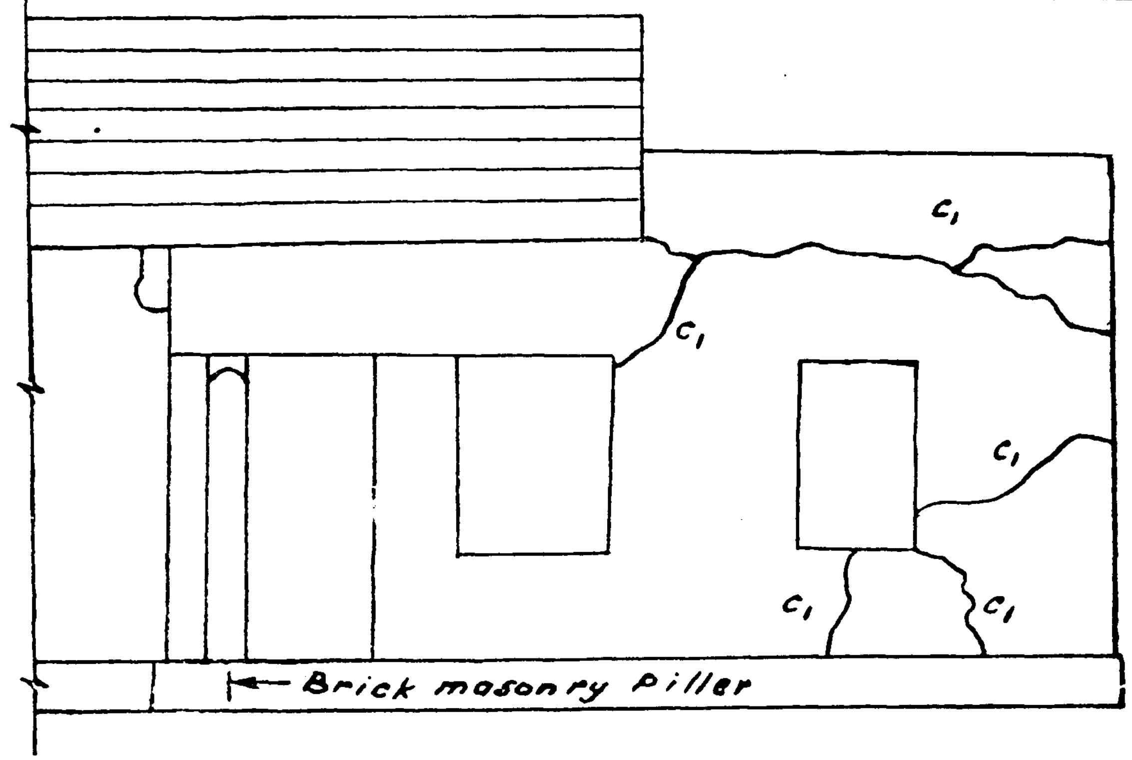

TYPICAL DETAILS OF UNDER REAMED PILES

Fig. .... 4.

PRASANNA VENKATESA PERUMAL WEAVERS
CO-OPERATIVE SOCIETY - MADURAI.

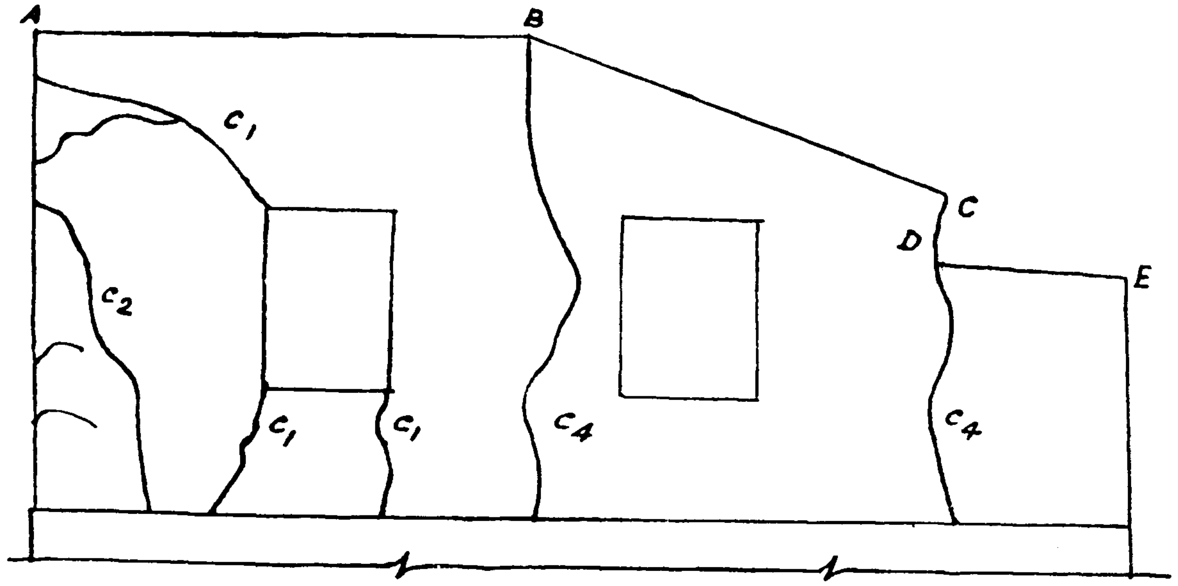


Sketch...1




FRONT ELEVATION

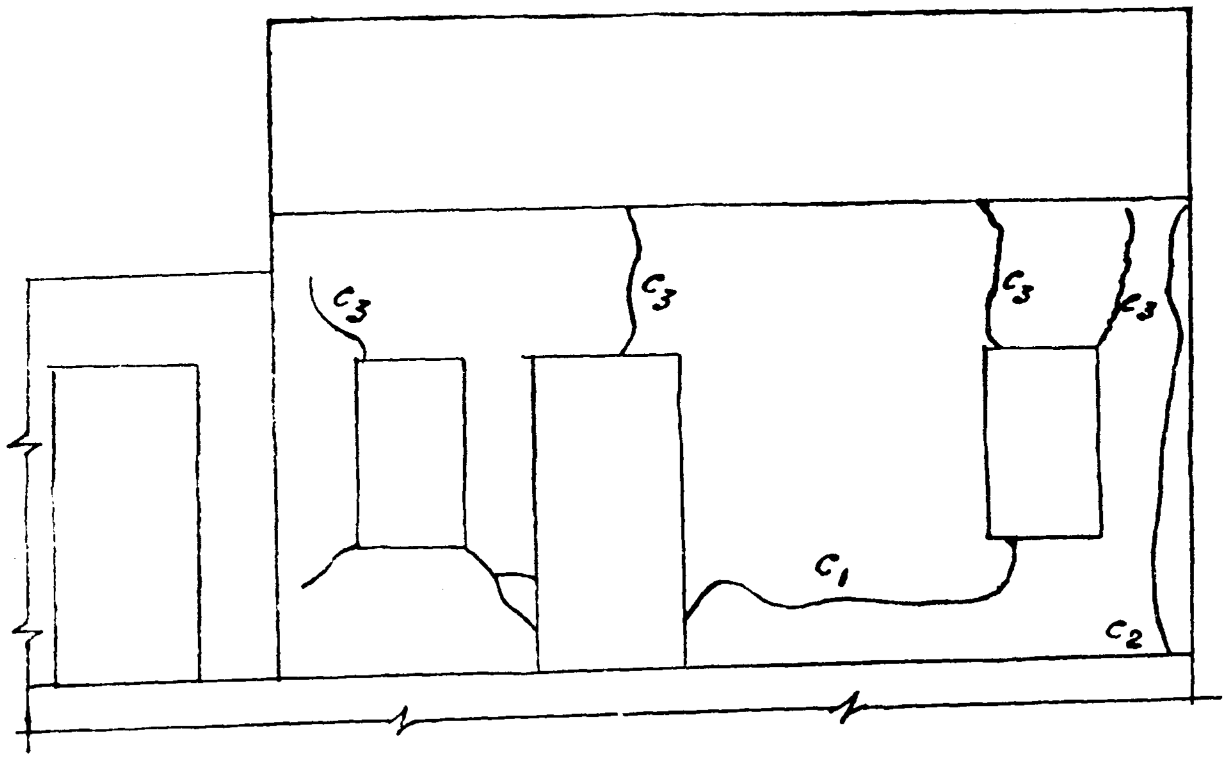
Sketch- 2.




END ELEVATION

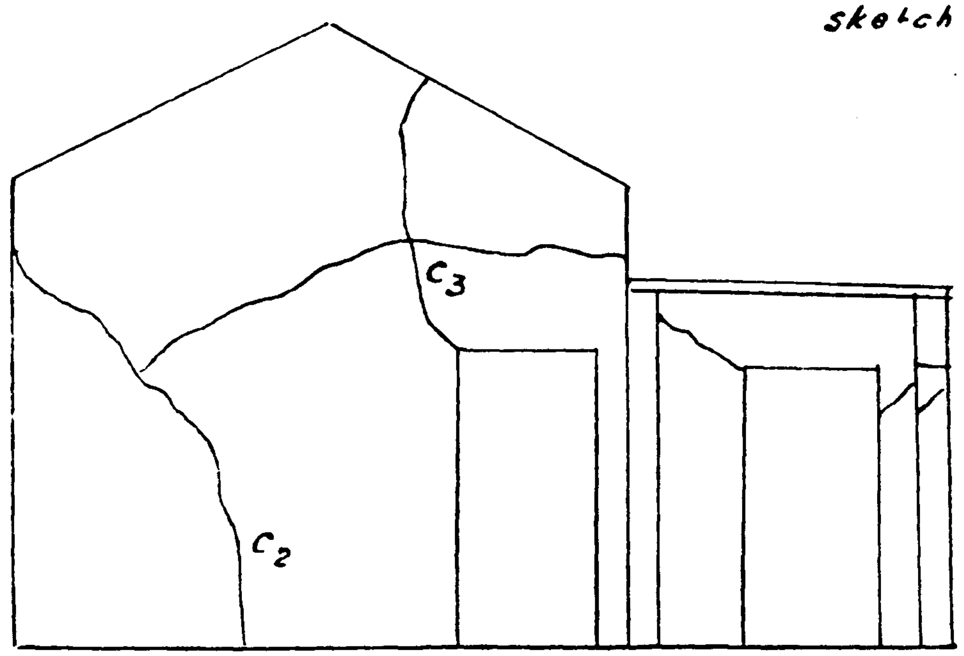
PRASANNA VENKATESA PERUMAL WEAVERS
CO-OPERATIVE SOCIETY- AVANIAPURAM NEAR
MADURAI

sketch-3




REAR ELEVATION

sketch 4.




WALL BETWEEN LIVING AND WORK ROOM

Prasanna Venkatesa Perumal weavers co-operative society—Avaniapuram Near Madurai.

# MASS CURVE FOR THE PROPOSED CHINNAR RESERVOIR PROJECT

**DHARMAPURI DISTRICT TAMIL NADU**

BY

Thiru S. KRISHNAMURTHY, Grad. I. E. (Ind.)*

## GENERAL :

1.1. A mass curve or a flow summation curve for a reservoir depicts the cumulative flows into and from the reservoir for several consecutive years. This mass curve is usually prepared for fixing the optimum storage capacity required for an impounding reservoir to meet a steady demand, which may be for an irrigation scheme, a water supply scheme, or a hydro electric scheme. In all such proposals, the storage capacities along with the possible regulations must be studied for several years including one or two cycles of dry years.

## CHARACTERISTICS OF A MASS CURVE :

1.2. A mass inflow or issue curve starts from the origin of coordinate axes the period being depicted on the abscissa and the cumulative flows on the 'Y' axes. The ordinate at any point on the mass curve represents the sum totals of flows upto that point. The slope of the curve at any point gives the rate of change of flow at that point. The mass curve will be steep when the rate of flow is large and flat when the flow is otherwise.

## DRAWING A MASS CURVE (MASS INFLOW CURVE).

1.3. (i) Several preliminary steps are required before a mass curve can be drawn. The first step is to collect the stream flow records for several years with those of one or two critical low flow years. A table with stream flow records is prepared showing the periods, inflows and cumulative inflows. The cumulative inflows are usually considered for ten to fifteen days if the period of study is for one or two years. Similarly, if the period of study is long, say, ten to twenty years, the time scale considered can be one month. Having tabulated the cumulative inflows, a mass inflow diagram showing the cumulative inflows for the periods considered can be drawn.

1.3. (ii) (MASS DRAW OFF CURVE): Similarly, cumulative demand or draw off from the reservoir will have to be prepared for the same periods, as considered for mass inflow diagram and the curve plotted. For uniform demand rates, such as for a water supply scheme, the demand curve will be a straight line, having its slope equal to the demand-rate. In such uniform demand-rates, the demand line is drawn tangential to the mass inflow curve at the peak point to get the optimum storage capacity required to maintain the uniform demand. Thus, if the reservoir were full at this peak point 'A' in chart No. 1, the tangent drawn will represent the required rate of inflow to meet the demand without depletion in the reservoir. But the rate of inflow being lesser than the rate of demand, the reservoir starts depleting just after 'A' and reaches maximum depletion at 'B'. Thereafter, the inflow rate being larger than the issue rate, the reservoir starts accumulating and is full again at C. Beyond 'C' the reservoir starts surplussing, as the rate of inflow is much larger than the demand rate and as the reservoir having become already full at 'C'. Thus, the greatest vertical distance 'BE' when measured to the scale of the ordinate is the minimum storage required to cope up the uniform demand. But, in actual practice, the demand cannot be uniform, say, for an irrigation scheme, and hydro electric scheme. It will very in an irrigation scheme, according to the crop requirements at different stages of growth, allowance to be made for useful rainfall, and the evaporation losses from fluctuating reservoir levels. In the event of such non-uniform demand rates, the procedure to be followed is to draw another curve for cumulative issues or draw off, corresponding to the cumulative inflow at that month. Here also, the maximum deviation of the mass issue curve from the mass inflow curve in the positive direction is the minimum storage required to maintain that non-uniform demand.

*Office of the Chief Engineer (Irrigation) Chepauk, Madras-5.

1.4. Sometimes, it may be necessary to find the maximum demand that can be expected from a reservoir of fixed storage capacity. This is done by marking an ordinate at the lowest point next to peak point in the mass inflow curve and joining the peak point to the ordinate marked earlier. The slope of this line represents the maximum rate of uniform demand that can be drawn from the fixed storage capacity.

## MASS CURVE FOR THE PROPOSED CHINNAR RESERVOIR PROJECT

2.1. The proposal is to form a reservoir of 500 m.cft. capacity, which aims at giving irrigation facilities to a new area of 1520 acres of lands; and to stabilise an extent of 2626 acres of lands fed by the river channel lower down. It is also proposed to give an uninterupted supply to a new extent of 354 acres under the Jertalavu tank, fed by a channel taking off from the river lower down. The following are some hydraulic features of the project.

| | |
|---|---|
| Catchment area | 240 square miles. |
| Annual average yield | 1820 m.cft. |
| Annual average drawal | 750 m.cft. |
| Proposed capacity | 500 m.cft. |
| Maximum flood discharge | 20040 cusecs. |

2.2. A mass inflow and issue curve have been drawn for the years 1941 to 1945, the first and the last years being periods of low flow years. This curve has been drawn by taking the cumulative inflows and issues and plotting them to a convenient scale. It is purposely assumed that the reservoir would have been empty during January 1941 though the reservoir were likely to have some initial storage, (about 330 m.cft.). In working out the monthly cumulative inflows, it has been noticed that in periods of good flows, the inflows were in the order of about 750 and 2200 m.cft. During such periods, to assess the actual inflows that could have been impounded and also to take into consideration the following dry periods, if any, it has been found necessary to assume a probable capacity of the reservoir and this is assumed as 400 m.cft. In case the maximum deficiency is greater than this four hundred million cubic feet, a second approximation has to be made and the process repeated. It can be seen from the table of cumulative flows that during September 1943 the cumulative demand is much higher than the cumulative inflow to the extent of 358 m.cft. Subsequently, there has been a large inflow of 2248 m.cft. during October 1943. As the reservoir is to be full during this month, the actual cumulative flow required to satisfy this condition is 1906+400 =2306 m.cft. This is equivalent to 765 m.cft. (2306 - 1541 being cumulative inflow of September 1943) actual apparent utilisable inflow against 2248 m.cft. realised and the balance 1483 m.cft. should be taken as imaginary surplus, the actual surplus being 1841 m.cft. Hence an utilisable inflow of 765 m.cft. has only been considered in cumulative calculations.

2.3 The irrigation period for this project is proposed from August to December and the demand is not uniform throughout the year. The slope of the mass issue curve is steep during irrigation season and flat during non irrigation periods, when the possible loss from the reservoir is evaper-colation only. It can be observed from the chart No. 2 that the reservoir is short of storage from August 1941 to September 1943 and the maximum shortage is 358 m.cft. Thereafter the reservoir starts building up storage by large inflow during October 1943. Thus from a study made for a cycle of five years, it is seen that the minimum storage capacity required to meet the demand is about 358 m.cft. However, the capacityof reservoir is fixed as 500 m.cft. considering various other aspects.

## ACKNOWLEDGEMENTS.

The author is indebted to Thiru P. K. Balakrishnan, Deputy Chief Engineer (Irrigation) for giving permission to utilise the records and for guiding him suitably. His sincere thanks are conveyed to other officers and collegues in the Office for their encouragement given in preparing this technical note.

The views expressed in this note are author's own and not necessarily that of the Department in which he serves.

## REFERENCES:

1. Chinnar Reservoir Project-Report.
2. Chief Engineer's Circular No. 585.45 B-6 d/29-7-47 (available in the report on " General Instructions for Investigations of Irrigation Projects")
3. Water Resources Engineering by RAY K. LINSLEY & JOSEPH B. FRANZINI
4. Water Resources Development by EDWARD KUIPER.
5. Rainfall and Runoff by E. FOSTER.



### TABLE OF CUMULATIVE FLOWS FOR THE PROPOSED CHINNAR RESERVOIR PROJECT

(Figures in Mc.ft.)

| Year and Month | Cumulative inflows | Cumulative issues | Total storage | Surplus | Remarks |
|---|---|---|---|---|---|
| 1941 | | | | | |
| January | 47 | 5 | 42 | — | |
| February | 72 | 10 | 62 | — | |
| March | 72 | 19 | 53 | — | |
| April | 73 | 30 | 43 | — | |
| May | 138 | 43 | 95 | — | |
| June | 139 | 52 | 87 | — | |
| July | 169 | 60 | 109 | — | |
| August | 172 | 239 | 67 | — | |
| September | 253 | 317 | 64 | — | |
| October | 318 | 509 | 191 | — | |
| November | 358 | 663 | 305 | — | |
| December | 436 | 712 | 336 | — | |
| 1942 | | | | | |
| January | 446 | 772 | 326 | — | |
| February | 454 | 773 | 319 | — | |
| March | 454 | 775 | 321 | — | |
| April | 454 | 777 | 323 | — | |
| May | 478 | 781 | 303 | — | |
| June | 509 | 785 | 276 | — | |
| July | 509 | 788 | 279 | — | |
| August | 637 | 851 | 214 | — | |
| September | 711 | 1010 | 299 | — | |
| October | 971 | 1098 | 127 | — | |
| November | 1109 | 1306 | 197 | — | |
| December | 1190 | 1458 | 268 | — | |
| 1943 | | | | | |
| January | 1270 | 1461 | 191 | — | |
| February | 1286 | 1464 | 178 | — | |
| March | 1286 | 1469 | 183 | — | |
| April | 1374 | 1479 | 105 | — | |
| May | 1432 | 1490 | 58 | — | |
| June | 1508 | 1498 | 10 | — | |
| July | 1515 | 1505 | 10 | — | |
| August | 1524 | 1720 | 196 | — | |
| September | 1541 | 1899 | 358 | — | |
| October | 2306 | 1906 | 400 | — | Actual inflow is 2248 Mc.ft. and this is restricted to 765 Mc. ft. Utilisable. |
| November | 2533 | 2090 | 400 | — | |
| December | 2589 | 2260 | 321 | — | |
| 1944 | | | | | |
| January | 2647 | 2273 | 374 | — | |
| February | 2682 | 2278 | 400 | 4 | |
| March | 2863 | 2287 | 400 | 176 | |
| April | 2919 | 2299 | 400 | 220 | |
| May | 2983 | 2313 | 400 | 270 | |
| June | 3101 | 2322 | 400 | 379 | |
| July | 3166 | 2330 | 400 | 436 | |
| August | 3320 | 2451 | 400 | 469 | |
| September | 3381 | 2512 | 400 | 469 | Actual inflow is 766 Mc. ft. and this is restricted to 61 Mc. ft. Utilisable. |
| October | 3458 | 2632 | 400 | 426 | |
| November | 3594 | 2769 | 400 | 425 | |
| December | 3686 | 2915 | 400 | 371 | |
| 1945 | | | | | |
| January | 3754 | 2920 | 400 | 434 | |
| February | 3807 | 2925 | 400 | 482 | |
| March | 3828 | 2934 | 400 | 494 | |
| April | 3850 | 2946 | 400 | 504 | |
| May | 3908 | 2960 | 400 | 548 | |
| June | 3970 | 2969 | 400 | 601 | |
| July | 4032 | 2977 | 400 | 655 | |
| August | 4089 | 3132 | 400 | 557 | |
| September | 4101 | 3266 | 400 | 435 | |
| October | 4133 | 3375 | 400 | 358 | |
| November | 4202 | 3529 | 400 | 273 | |
| December | 4258 | 3704 | 400 | 154 | |

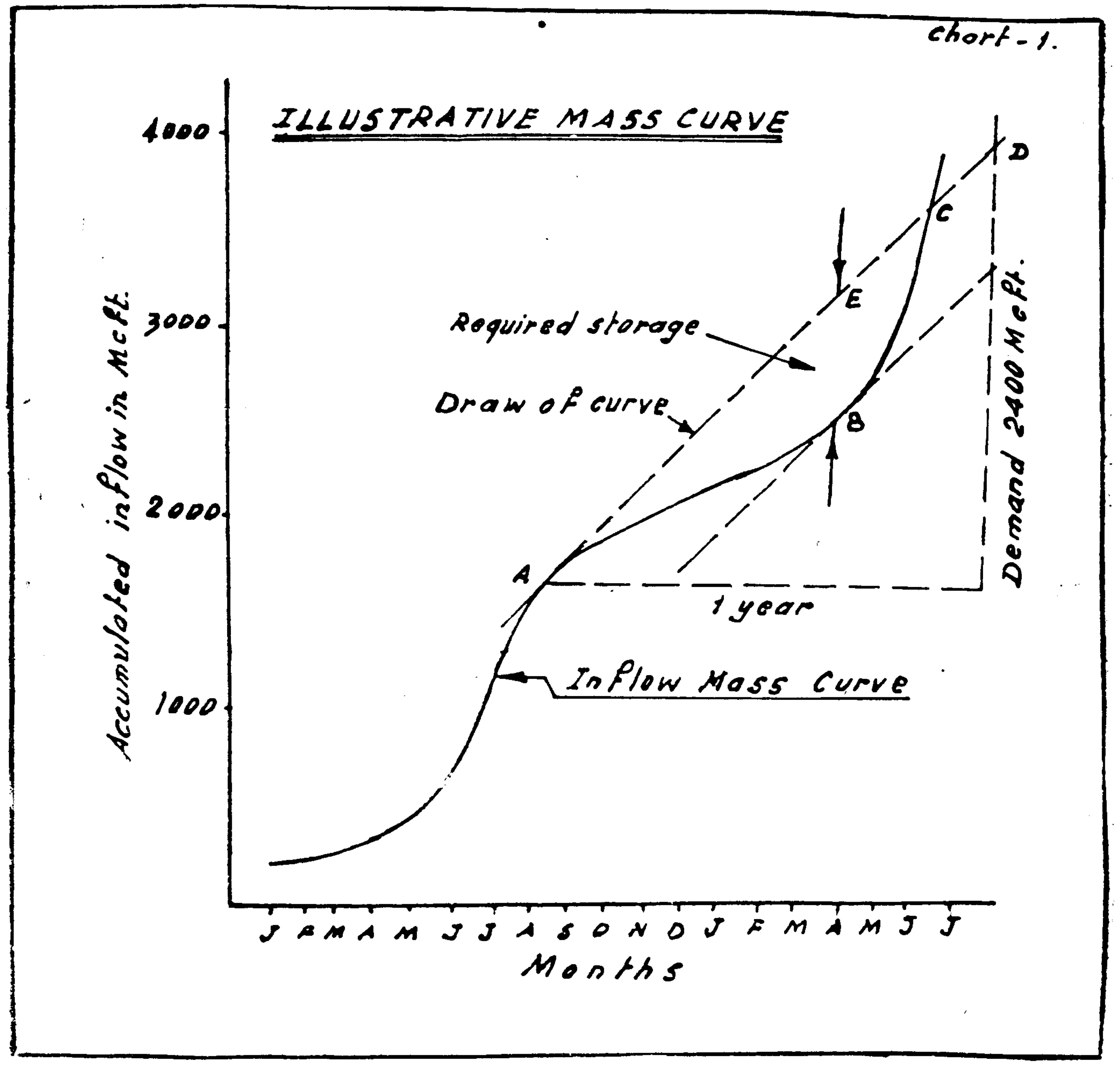
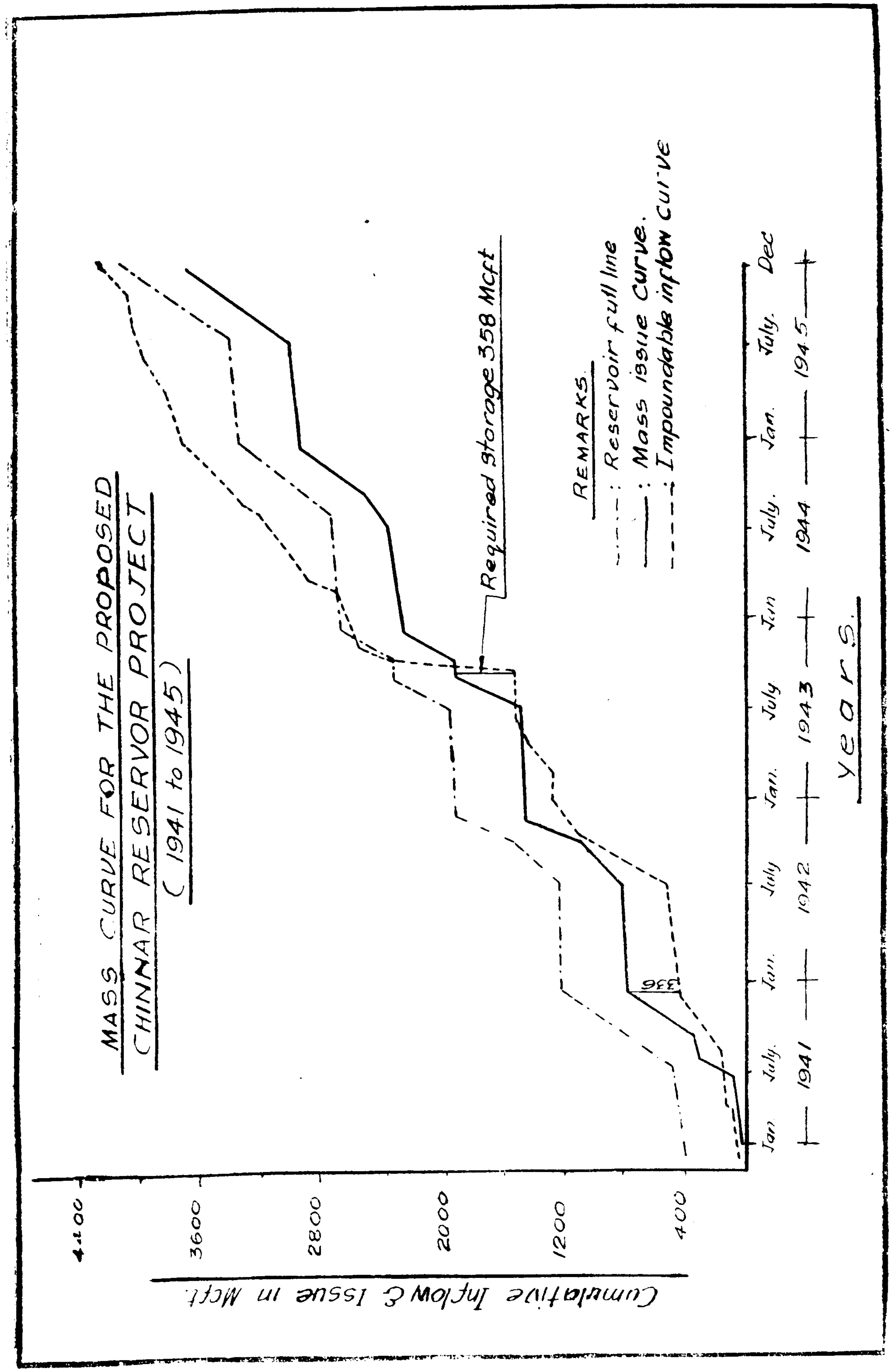


Chart - 1.
ILLUSTRATIVE MASS CURVE
Accumulated inflow in Mcft.
4000
3000
2000
1000
Required storage
Draw of curve
Inflow Mass Curve
1 year
Demand 2400 Mcft.
A
B
C
D
E
J F M A M J J A S O N D J F M A M J J
Months


MASS CURVE FOR THE PROPOSED
CHINNAR RESERVOR PROJECT
( 1941 to 1945 )
Required storage 358 Mcft
336
REMARKS.
Reservoir full line
Mass issue Curve.
Impoundable inflow curve
Cumulative Inflow & Issue in Mcft.
4400
3600
2800
2000
1200
400
Jan July Jan July Jan July Jan July Jan July Dec
1941 1942 1943 1944 1945
Years.

# NEWS FORM ABROAD

**(1) Raising of Plover Cove dams will increase Hong Kong's Water supply**

Work has begun to raise Hong Kong's Plover Cove dams and so increase the reservoir's storage capacity from 30,000 million to 50,500 million gallons. The crests of the main mile and-a-quarter long dam and two subsidiary dams, now about 40 ft. above the low-tide level, would be raised by 12 ft.

In addition, a small dam will be constructed to seal off a low saddle in the Tai Mei Tuk peninsula, and a siphon spillway will be constructed on the site of the present spillway. Soft filling materials and rock for the raising of the dams will be taken from borrow areas adjacent to the road on the north coast of Tolo Harbour and from an existing quarry at Turret Hill.

Other works connected with the project have already begun. These include the installation of additional pumps at the Tai Po Tan and Sha Tin pumping stations and extensions to the treatment works at Sha Tin for additional water becoming available from Plover Cove after the dams have been raised. The raising of the dam is expected to be completed by summer 1973.

"Civil Engineering and Public Works Review"
September. 1970.

**2. Surface stabilisation by Hydraulic Seeding.**

B. W. D. (Hydraulic seeding) recently carried out a surface stabilisation project for British Rail. Steep cuttings in block chalk have over a period of time produced serious problems for British Rail in-as-much-as chalk weathers, causing boulders of chalk to become dislodged and fall on to passing trains thus creating serious and costly delays to traffic at the entrance to a tunnel.

The method adopted to overcome the problem has been to drive steel pins at approximately 1.5m centres into the surface of the cutting on which to hang sprayed glass rovings stabilised in site with bitumen emulsion Beneath the grass mat hydraulic seeding was carried out, incorporating a new product of British Gypsum Limited, namely Agrahyd.

**3. Inflatable cylindrical void former :**

Contiduct is an inflatable cylindrical void former with a high re-use factor. It is a specially reinforced, tough Rubber membrane available in tubular form from 300 mm to 4500 mm. The inner membrane ensures air tightness, the reinforcing ensures a high degree of shape retention throughout its entire length and the outer skin is designed to withstand normal rough usage obtained on site. The ends of the cylinders are closed using scientifically designed endcones which prevent diametrical draw-in on inflation.

Up to a short while ago it was not possible for technical reasons to construct a Contiduct of such a large diameter, however, due to recent advances in technology the Ductube company Limited have been able to construct a 3.7 m diameter Contiduct which is now in the process of being despatched to Greece.

**4. Portable Laser Beam :**

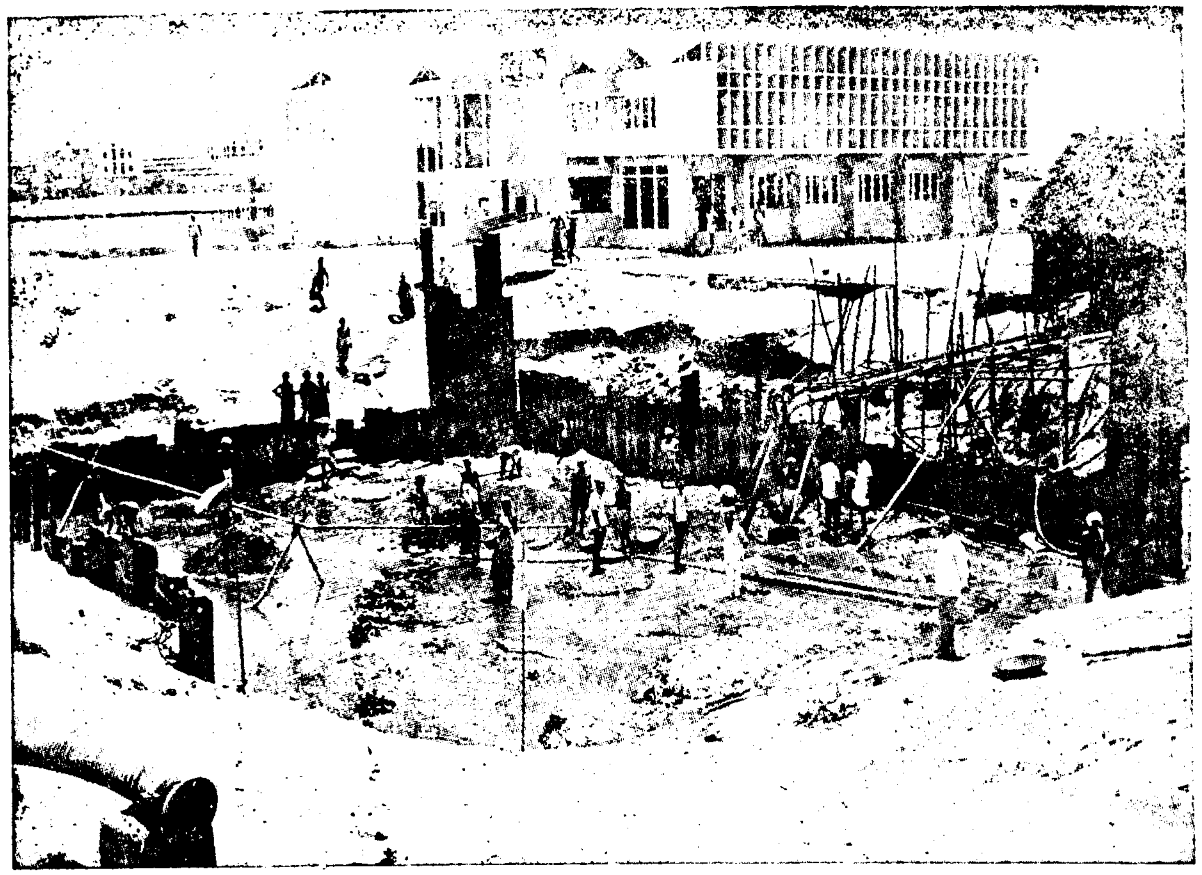
A simple and Safe Laser Projector to give an accurate visible reference line for construction, has been developed by Elliott Automation Radar Systems Limited (EARS). Called Laserline, this portable, battery powered device can project a beam of red light, which can be picked up with a simple reflector, even in sunlight, at any range upto a quarter of a mile or more.. The beam is not disturbed by the passage of vehicles or equipment.

Once setup on its tripod using its telescopic sight, it can be left unmanned and the beam detected at any point at which work is in progress, eliminating the necessity of a second man at the instrument such as is the case with a theodolite or level.

The laserline forms an accurate reference line for laying pipes at precise gradients, constructing tall or long buildings, observing movement in tunnels and mining galleries and, by traversing the beam, precise planes can be defined for excavating and laying roadways and runways. It has already been used as a precise vertical reference for the accurate and rapid raising of the mould shuttering in the continuous pouring operation in the construction of tall concrete silos.

The future potential of this device looks interesting and it shows most promise in the construction and civil Engineering fields, in mining, pipe laying and even agriculture.

" The consulting engineer " September 1970.



DEEP DIVING POOL IN PROGRESS

SWIMMING POOL WITH DIVING BOARD IN THE REAR

# INVITATION TO READERS

Kind attention of the Engineers of the Madras Public Works Department and readers of the journal elsewhere is drawn to the meagre response to repeated requests for contribution of articles to the journal.

May we once again emphasise the two-fold objective of the journal, viz., to promote the interest of engineers and aiding their specialised work by placing before them the many problems of technical importance and ways and means of solving them, and secondly, opportunities being scarce for engineers to express their own experience in solving practical problems of design and execution, we wish to add that this journal has been the only medium of expression for those in the field. It is earnestly desired that the Engineering fraternity will come forward in large numbers to utilise this medium to its best advantage by offering their valuable contribution.

The scope of the articles to the journal need not be rigorously confined to the subjects on irrigation alone. Articles which have a direct bearing on subjects given below are also welcome :—

(1) Technical aspects of foundation soils and foundation Engineering.

(2) Auxiliary works in the field of irrigation and other water-retaining structures and works.

(3) Practical solution to hydraulic problems in canals and distribution systems with reference to their maintenance.

(4) Problems of soil erosion and conservation and reclamation of land.

(5) Practical silt problems in rivers and reservoirs.

(6) Problems of sanitation and public health and possibilities of utilisation of sewage.

(7) Inland waterways and navigation, fishing schemes and protection of land against sea erosion, etc.

(8) Problems of drainage in deltaic tracts and entrance of rivers into the sea.

(9) Run-off studies in catchments and evaporation studies in reservoirs.

(10) All allied subjects.

The Executive Engineers of the Madras Public Works Department are specially requested to encourage their Subordinate Officers to come forward with articles and enable the journal to flourish and serve the common cause of the Engineers of the State.

SOLDEN & CO. (PRINTERS) MADRAS-5.





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