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"DESIGN AND CONSTRUCTION OF PRESTRESSED CONCRETE BRIDGE ACROSS RIVER PALAR"

By

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CONTENTS

SYNOPSIS

The Paper describes the construction of prestressed concrete bridge across River Palar at mile 38/3-5 of the Madras-Dindigul Road. The bridge consists of 23 spans of 90 ft clear each and is founded on wells. The superstructure for each of the spans consists of an R. C. slab 6 in. thick cast *in situ* over four precast prestressed U-shaped girders each 95 ft 10 in. long.

Details of design, precasting of the girders, prestressing and launching the girders weighing 60 tons each from the casting yard to their positions over the piers are given.

1. INTRODUCTION AND LOCATION

Madras-Dindigul Road, the National Highway connecting Madras to the southern districts of the State, crosses River Palar at mile 38/3-5 from Madras. The river Palar rises in the Kolar District of Mysore State and joins the Bay of Bengal about 10 miles from the site of the bridge. Plate I shows the course of the river and the site. There had been a bed level causeway at the site and the want of a bridge was keenly felt as interruptions to traffic caused great inconvenience to the public.

2. DESIGN DISCHARGE

The discharge of the river was worked out taking 7 cross sections of the river both upstream and downstream of the site. The average of the discharges for two cross-sections, one 5,000 ft upstream and the other 500 ft downstream, worked out to 2,53,972 cusecs. The discharge at the railway bridge situated four miles downstream for the 1946 floods gave a figure of 2,67,957 cusecs. Computations were also made for the flood discharge over the anicut situated 60 miles above the site and for the tributaries, Cheyyar and Soyyar, which join the river between the anicut and site of the bridge. This gave a discharge of 3,28,000 cusecs. The highest figure, viz., 3,28,000 cusecs, was adopted for the design.

The bridge is from bank to bank with a lineal waterway of 2,070 feet.

3. TYPE OF BRIDGE

Borings taken at the site of bridge revealed sandy soil to a depth of about 12 ft overlying a layer of hard stiff clay ranging from 10 to 30 ft in depth. Below this strata disintegrated rock was found, Plate II. It was first proposed to adopt 27 spans of 60 ft clear with continuous Tee-beams and 4 spans of 100 ft with bowstring girders. The alternative design of one of the tenderers for a prestressed concrete superstructure with 90 ft spans was accepted after slight modifications. As prestressing was a new technique the contract agreement included a condition that a test to destruction should be conducted at contractor's cost.

4. GENERAL DESCRIPTION OF THE BRIDGE

4.1. Sub-Structure

The bridge is 2,202 ft between the faces of the abutments and consists of 23 spans of 90 ft clear each. The piers are of plum concrete 6 ft wide at bed block level and are 7 ft wide at capping slab level with a batter of about 1/32. The foundations for each pier consist of two circular wells of 12 ft external diameter with 8 ft plug holes. The clear space between the wells is 5 feet. The steining is 2 ft thick and is of cement concrete 1 : 3 : 6, Plate III. The wells are provided with R. C. curbs with cutting edge and were sunk to an average depth of 32 feet. The bottom 5 ft of the well is plugged with cement concrete 1 : 2 : 4. Sand filling is done between bottom plug and top plug of concrete 2-ft thick (cement concrete 1 : 4 : 8). The wells are provided with R. C. capping slab 27 in. thick.

4.2. Superstructure

The deck consists of 4 pre-cast prestressed girders 95 ft 10 in. long with a 6 in. thick R. C. slab cast *in situ* on top. The girders are of cellular construction 61 in. deep and 41 in. wide. Precast diaphragms 6 in. thick are spaced at 9 ft 6 in. centres with two massive end blocks 4 ft 10 in. wide giving 9 bays of U-shaped units, Plate IV. The soffit slab is 6 in. thick and side webs are 5 in. thick. Each U-girder is prestressed according to Magnel Blaton system with 256 wires of 5 mm. diameter each. In one girder there are eight cables of 32 wires each. Four cables are taken straight and anchored at end blocks. The other four cables are taken in a parabola and anchored at 13 in. from top of the U-sections. These parabolic cables reduce the end shear and also keep down the eccentricity so that no tension occurs in concrete.

The roadway of the bridge is 22 ft wide with 9 in. by 15 in. curbs on either side. The wearing coat is of 1 : $1\frac{1}{2}$: 3 cement concrete with cross camber for drainage.

The girders are provided with a rocker bearing for the free end and rest on a mortar pad 6 in. by 1 in. section at the fixed end.

The railings consist of R. C. panels with R. C. posts.

Auxiliary wings are provided to give architectural effect and incidentally they reduce the length of the main wing.

Approaches are taken in a gradient of 1 in 30. Horizontal curves with radius of 1,000 ft with necessary transition curves have been provided.

5. SPECIFICATION OF MATERIALS

5.1. It has been the general practice in the department to specify concrete by volume proportions. It was felt that a specification based on a rational design for the mix was necessary as a high working stress of 1,500 lb per sq. in. was adopted in the design of prestressed concrete girders. Hence strength requirements were specified in addition to the nominal mix.

No special grading was adopted; periodically sieve analysis of the aggregate was conducted, Fig. 1 in Plate V.

Sand was taken from the river bed. The Palar sand contains fairly coarse angular grains. A typical bulk graph is shown in Fig. 2 in Plate V.

Batching for concrete was done by weighing sand. Aggregate was measured by volume. One bag of cement was taken to weigh 110 lb and water-cement ratio was regulated. Though laboratory tests indicated water-cement ratio of 0.42 to 0.45 could be used to give a concrete strength of 5,000 lb per sq. in. at 28 days, on the job a ratio of 0.4 was adopted.

For R. C. works, a nominal mix of 1 : 2 : 4 was adopted. Specified strength was 3,000 lb per sq. in. at 28 days. The batching was done by measuring volumes.

In order to ensure high quality work, a penalty clause was stipulated in the agreement to the effect that in case the concrete used did not yield the specified strength, recovery at twice the quoted rate would be effected from the contractor. The strengths specified at 28 days and the design stresses adopted for the various kinds of concrete are given below:

Specification of concrete mix	Specified strength at 28 days on 6 in. cubes lb per sq. in.	Design stress lb per sq. in.
Prestressed concrete work 1 : 1½ : 3	5,000	1,500
Cast in situ diaphragms and deck slab 1 : 2 : 4	3,000	950
Capping slab and bed block 1 : 2 : 4	3,000	750

5.2. High Tensile Steel

High tensile steel wires, were obtained by the contractors from abroad and these were periodically tested. As high tensile wire does not show any definite yield point, "Proof stress" was determined for 0.1 per cent permanent strain, Plate VI. As

recommended by Prof. Magnel, steel wires were tested to see if the working stress (140,000 lb per sq. in.) adopted was less than 0.8 of proof stress or 0.6 of ultimate stress. The steel that was supplied by the contractors was very satisfactory.

5.3. Mild Steel

Tested mild steel was used. The usual working stresss of 18,000 lb per sq. in. was adopted.

5.4. Cement

Cement supplied by Messrs Dalmia Cement Co. was used. This was periodically got tested at Government Test House, Alipore, Calcutta, for Indian Standards Institution Specification.

6. DESIGN

The salient principles of design of the prestressed girder are indicated below.

6 1. A prestressed girder has to satisfy the following conditions:

Top fibre of Girder

(a) On the application of prestress the tensile stress in the concrete under dead load must not exceed the permissible tensile stress in the concrete.

(b) When losses due to plastic flow and shrinkage have taken place, the permissible compressive stress in concrete under superimposed loads must not be exceeded.

Bottom Fibre

(a) Immediately the prestress is established compressive stress should not exceed the permissible value.

(b) Under superimposed load, when losses have taken place, tension in the girder should not exceed the permissible value.

For a beam under prestress from the conditions given above it is possible to establish the relationship

$$(I-n)\frac{Pi}{A}\left(\frac{ey_b}{r^2}+1\right)+cab<C+ct \qquad \ldots\ldots(1)$$

where Pi = Prestress applied

n = Proportion of prestress that remains permanently

A = Area of section

e = Eccentricity

y_t, y_b = Distance of top and bottom fibres respectively from the neutral axis of section

C_{dt} = Stress due to dead load at top

C_{at} = Stress due to superimposed load at top

C_{db} = Stress due to dead load at bottom

C_{ab} = Stress due to superimposed loads i.e., additional loads applied after the prestress at bottom

C = Permissible compressive stress

C_t = Permissible tensile stress

r = Radius of gyration of the concrete section

Since $Cab = \frac{M_a y_b}{I}$ where M_a is the bending moment due to additional loads applied after prestress is established. Expression in (1) can be written

$$\frac{I}{y_b} = \frac{M_a}{C + ct - (I - n)\frac{Pi}{A}\left(\frac{ey_b}{r^2} + 1\right)} \qquad \text{......(2)}$$

for the limit $n = 1$, $\frac{I}{y_b} = \frac{Ma}{C + ct}$

Thus a prestressed girder has to be designed only for live load bending moment and as for dead load it "carries itself"*. The bending moments at the centre of the span due to the precast beam, deck slab, hand-rails, etc., and the live load are given below:

Load per Rft.	Bending Moment inch lb
Precast girder including diaphragms 1192 lb	... 15,400,000
Deck slab 522 lb	... 6,800,000
Wearing coat, Hand rails and Kerbs 1264 lb	... 4,100,000
Live Load	... 12,200,000

* Prestressed Concrete by G. Magnel, Concrete Publications Ltd., 14 Dartmouth Street, London.

The section chosen for the girder is shown in Fig. 1.

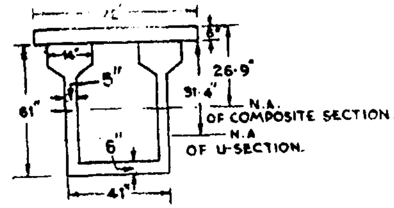

Fig. 1

The properties of the U-shaped precast, prestressed section are:

Area of Section	... 958 sq. in.
Moment of Inertia	... 4,39,794 in.^{4}
Section Modulus $Z_1 = \frac{I}{y_t}$	... 14,000 in.^{3}
Section Modulus $Z_2 = \frac{I}{y_b}$	... 14,900 in.^{3}

After the 6 in. deck slab is cast the composite section will have the following properties:

Area of Section	... 1,390 sq. in.
Moment of Inertia	... 7,93,794 in.^{4}
Section Modulus $Z_1 = \frac{I}{y_t}$	... 29,500 in.^{3}
Section Modulus $Z_2 = \frac{I}{y_b}$	... 19,800 in.^{3}

While prestress is applied only the U-portion of the girder will be effective. When the *in situ* slab is cast, the composite section will be effective.

The bending moment due to loads applied after the prestress is established i.e., live load, slab, wearing coat and hand rails etc. is 2,31,00,000 in.-lb.

Minimum value of $\frac{I}{y_b} = \frac{2,31,00,000}{1,500} = 15,400$ in. units.

Section adopted gives 19,800 in. units.

6.2. Determination of Prestress :

The prestress moment depends upon prestress as well as eccentricity. The chosen values of eccentricity and prestress must be such that the conditions mentioned below must be satisfied. In the general case, $e > \frac{r^2}{y_t}$

Condition 1.

At prestress, the top fibre should not be subject to tension with the combination of dead load.

$$\frac{Pi}{A}\left(\frac{eyt}{r^2}-1\right)-Cdt=0 \qquad \ldots\ldots(1)$$

Condition 2.

After losses due to shrinkage and creep, under superimposed load the stress in the top fibre must not exceed allowable compressive stress.

$$-\frac{n\,Pi}{A}\left(\frac{eyt}{r^2}-1\right)+Cdt+Cat=C \qquad \ldots\ldots(2)$$

Condition 3.

At prestress, the stress in the bottom fibre taking into account dead load must not exceed the permissible value

$$\frac{Pi}{A}\left(\frac{ey_b}{r^2}+1\right)+Cdb=C \qquad \ldots\ldots(3)$$

Condition 4.

After losses and allowing for the dead load and superimposed loads the bottom must not be subject to tension.

$$-\frac{n\,Pi}{A}\left(\frac{ey_b}{r^2}+1\right)+Cdb+Cab=0 \qquad \ldots\ldots(4)$$

The prestress e chosen must simultaneously satisfy the conditions above. The four expression can be rearranged in terms of $\frac{A}{Pi}$ and by putting $e=0$ the value for A/Pi for the 4 conditions give the result

(1) $\frac{A}{P_i} = -\frac{1}{Cdt}$ (2) $\frac{A}{P_i} = -\frac{n}{Cdt-Cat-C}$

(3) $\frac{A}{P_i} = \frac{1}{C+Cdb}$ (4) $\frac{A}{P_i} = \frac{n}{Cdt+Cab}$

Therefore evaluating the above expressions simultaneously it is possible to determine $\frac{A}{P_i}$ satisfying all the four conditions. A simple method of solving is by drawing graph for the four

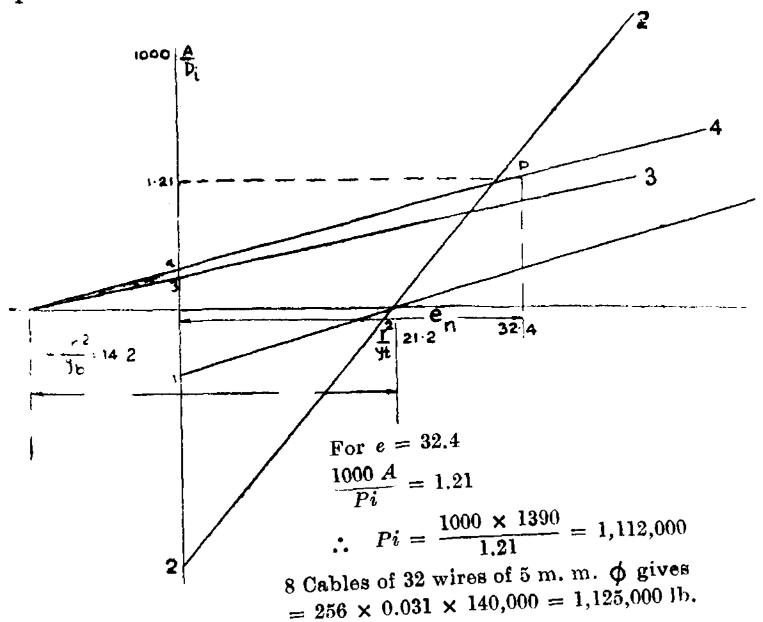

Fig. 2

conditions as shown in Fig. 2. It is obvious that the chosen value of $\frac{P_i}{A}$ must be within the area bounded by lines 2, 3 and 4 to the left.

Cdt Stress due to dead load at top 1100 + 486 = 1,586 lb per sq. in.

Cat Stress due to superimposed load at top : 429

C Allowable compressive stress = 1,500 lb

Cdb Stress due to dead load at bottom = 1030 + 457 = 1,487

Cab Stress due to live load at bottom = 822

Stresses are determined by using the fundamental bending formula $f = \frac{My}{I}$

Stresses are given below :

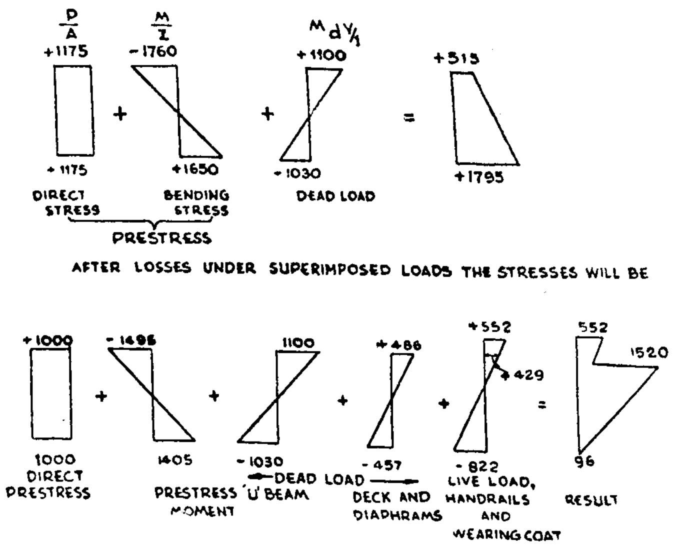

Fig. 3

The prestress can be calculated analytically also by making use of equation for line 4 in Fig. 2 as the chosen point P lies on this line.

$$P_i = \frac{(Cdb + Cab)\,A}{n\left(1 + \frac{eyb}{r^2}\right)}$$

$$= \frac{(1487 + 822)\,1390}{0.85\left(1 + \frac{32.4}{14.2}\right)} = 1{,}111{,}040 \text{ lb}$$

In practice 8 cables of 32 wires will be used

Therefore prestress $= 256 \times 0.0314 \times 140000$

$= 1{,}125{,}000$ lb.

It is interesting to investigate the factor of safety for first crack and for failure of prestressed concrete girder.

Moment at 1st crack = total moment $+ f\,cr\,\frac{I}{yb}$ where

fcr = Stress at which tensile cracks appear in concrete. During the ultimate load *test conducted on a test beam it was observed that cracks appeared at about 150 lb per sq. in.

$$38500000 + 150 \times \frac{793794}{40.1} = 41{,}476{,}707 \text{ in lb.}$$

This means there is a factor of safety of 1.70 against cracking.

Failure at ultimate load is preceded by plastic elongation of wire and crushing of concrete.

Ultimate resisting moment of section.

$M_u = A_s\,f_u\,ja.$

$256 \times 0.0314 \times 220000 \times 0.9 \times 56 = 89{,}100{,}000$ in lb.

This represents 2.3 times live load and dead load B.M. taken together or more than 5 times the live load B.M. alone.

6.3. Design of End Block

The end blocks which have to bear a reaction of 500 tons are designed as per the principles laid down by G. Magnel.*

The Maximum stresses in an end block occur at the moment when all the cables have been stressed.

The stresses are :

(i) Bending stresses due to the horizontal moment produced in the block by the anchorages pressing on one end and the U-shaped section at the other.

(ii) Horizontal shear stresses induced by the above forces.

(iii) Vertical shear—i.e., the difference between the vertical shear from dead load and negative shear due to prestressing cables.

(iv) Compressive stresses acting on a vertical plane due to cable force.

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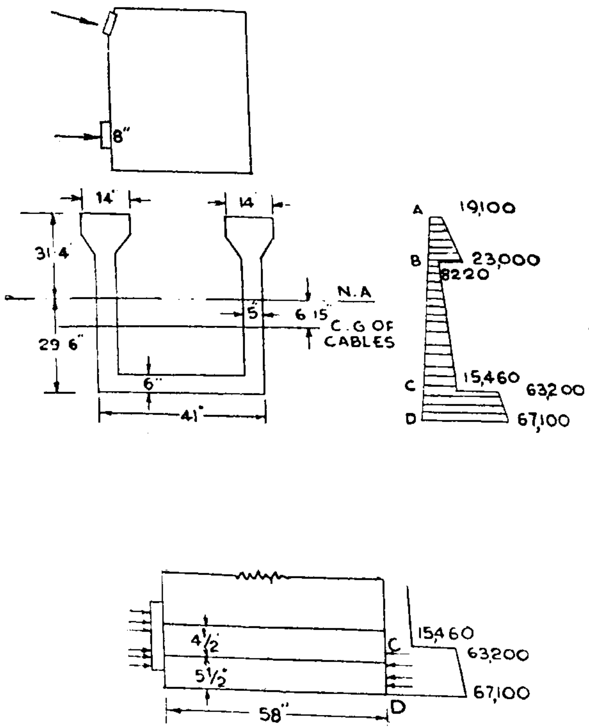
* Nambiar K. K. and Raj P. V. Testing to Destruction a 96-foot Prestressed Concrete Bridge Girder, Journal of the Indian Roads Congress Vol. XVII-3, January, 1953.

* Prestressed Concrete by G. Magnel, Concrete Publications Ltd., 14 Dartmouth Street, London.



AB=9 in.

Fig. 4

(1) Tension due to horizontal bending $128 \times 0.0314 / 140{,}000 = 562{,}000$ lb.

Pressure per inch of distribution plate $56200/8 = 70200$ lb.

At face of end block

$A = 958$ sq. in.

$I = 439794$ in.^{4}

$P = 11{,}25{,}000$ lb

$e = 6.15$

$11{,}25{,}000 \times 6.15 = 69{,}20{,}000$ in. lb.

In Fig. 4 Stress at A $= \dfrac{1125000}{958} - \dfrac{6920000}{439794} \times 31.4$

$= 1175 - 488 = 687$ lb per sq. in.

B $1175 - \dfrac{6920000}{439794} \times 22.4 = 822$ lb per sq. in.

C $1175 + \dfrac{6920000}{439794} \times 23.6 = 1546$ lb per sq. in.

D $1175 + \dfrac{6920000}{439794} \times 29.6 = 1640$ lb per sq. in.

Force on block per inch height at

A $687 \times 28 = 19100$ lb nearly

B $822 \times 28 = 23000$ lb

$822 \times 10 = 8220$ lb

C $1546 \times 10 = 15460$ lb

$1546 \times 41 = 63200$ lb nearly.

D $1640 \times 41 = 67100$ lb nearly.

Horizontal moment : The forces acting are as indicated above.

B.M. $67100 \times 6 \times 7 + 15460 \times 4 \times 2 - 70200 \times 6.25 \times 3.125 = 15{,}74{,}000$ in.-lb.

According to Magnel, tensile stress occurs at centre of the section and is equal to $\dfrac{5\ M}{bd^2}$

$$\frac{5 \times 1574000}{(41 - 4 \times 2\frac{1}{2})\,58^2} = 76 \text{ lb per sq. in.}$$

(2) Horizontal shear stress :

$67100 \times 5.5 - 70200 \times 1.75 = 247000$

Shear stress $= \dfrac{1.25s}{bd}$

$$\frac{1.25 \times 24700}{41 \times 58} = 172 \text{ lb per sq. in.}$$

(3) Vertical shear stress:

Shear due to dead load 50,800

Weight of end block 6,050

56,850

Negative shear due to

Prestress 58,200

Net shear -1350 lb

Shear stress $\frac{1350}{58 \times 41} = 0.6$ lb

(4) Compressive stress in a vertical plane

At the centre of the end block eccentricity is 3.6 in.

B. M. 1125000 × 3.6 = 4050000. Stress *C* at centre of lower ducts

$$\frac{1125000}{61 \times 41} + \frac{6 \times 4050000}{41 \times 61^2} = 450 + 159 = 609 \text{ lb per sq. in.}$$

Summary of stresses:

(i) Tension due to horizontal bending 76 lb

(ii) Horizontal shear stress 172 lb

(iii) Compression on a rest plane 609 lb

Principal tensile stress

$$\frac{609-76}{2} - \sqrt{\left(172^2 + \frac{(609+76)^2}{2}\right)}$$

267 - 384 = -117 lb per sq. in.

Total tension per foot of end block 117 × 31 × 12 43500 lb.

Area of steel per foot at 24000 lb per sq. in. stress 43500/24000 = 1.87 sq. in. 5/8 in. dia. rods at $4\frac{1}{2}$ in. centres are used.

7. CONSTRUCTION OF FOUNDATIONS AND SUBSTRUCTURE

7.1. Wells

Two permanent theodolite stations were constructed for aligning the bridge axis. The bed of the river happened to be dry for many months during construction and hence no special problems were encountered in laying the well curbs. Mild steel sheets 1/8 in. thick, stiffened with M. S. angles, were used for the shuttering of the R. C. kerbs. The inner face of the kerb is provided with a champher of 1 ft 6 in. in 3 ft 6 inches. The cutting edge is of M. S. angles 6 in. × 6 in. × $\frac{1}{2}$ in. See Plate VII. These angles were welded to the bond rods of the steining. Cement concrete 1 : 2 : 4 was used. See Photos 3 and 4.

7.2. Steining

Reinforcement of the steining consisted of 22 numbers of 1 in. diameter and $\frac{3}{8}$ in. diameter hoop rods at every 2 ft centres. The concrete used was 1 : 3 : 6 with $1\frac{1}{2}$ in. gauge granite metal. Steinings were raised in stages of 4 ft using steel shutterings. To facilitate fixing of the shuttering for the next stage, holes were left 6 in. below the top of each stage so that the shuttering could be fixed with an overlap of 6 inches. These holes were grouted with mortar after casting was over. Timber distance-pieces were used to maintain the thickness of steining at 2 feet. External timber struts were used to prevent any bulging.

7.3. Sinking

Sinking was done by open dredging up to about 20 ft depth. Two 6 in. centrifugal pumps and sump pumps operated by compressed air were used for the purpose. This was possible only when there was no blow of sand. When blows oucurred under-water sinking was resorted to. Diving helmets were used for the purpose with the aid of one Atlas Diesel Air Compressor of 110 cu. ft. capacity. One compressor could feed about 8 helmets at depths of 30 to 40 feet. First a pit of about 2 ft was made in the centre and then earth from under the cutting edge was cleared. Care was taken to see that the clearance was uniform on all sides. As the dredge hole was 8 ft in diameter, two or three divers could work comfortably. Each sinker could work in water continuously for about 4 hours. After the underside of the cutting edge was cleared Photo 5 a small blast using gelignite was given to the water in the well to facilitate sinking of the well. This eased the skin friction and well sunk easily.

At depths greater than 20 ft isolated big boulders embedded in stiff clay and disintegrated rock were met with during sinking.

When these were found in the area of the plug hole, they were hauled up by using a winch and wire rope. Often boulders beneath the cutting edge retarded progress. In such cases, holes were drilled with pneumatic drills under water and the boulders were blasted by means of gelignite. The fuse of the charge was connected to a battery and ignition was effected by connecting the terminals from outside the well; only half charges were used to prevent damage to the steining during blasting.

It was felt that resting wells on independent boulders embedded in clay was not desirable. So they were sunk through the layer of clay mixed with boulder and were plugged only when good seating on boulders on all sides or disintegrated rock was met with. While sinking the up stream side well of pier 3, a very big boulder embedded in disintegrated rock 2 ft below cutting edge and extending to nearly half the area of the well was met with and it was found that the boulder extended outside the cutting edge. When a pit 8 ft deep was made, two other boulders were also found. The big boulder was blasted and efforts were made to remove it. After blasting 2 ft of the boulder it was

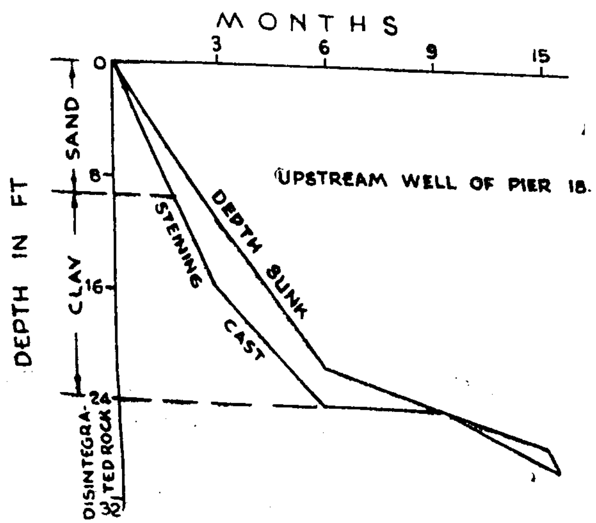


Fig. 5

found that further blasting might damage the well. It was observed that the boulder has resisted the shocks due to the blasting. As the well was resting on boulders on other sides also and

Photo 1. Site of the bridge. Existing causeway can be seen on the left

Photo 2. Existing causeway and piers of the bridge are seen on the right

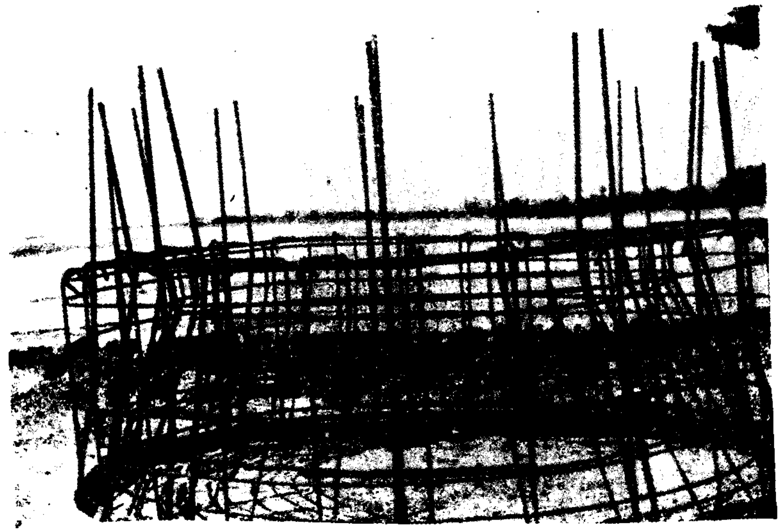


Photo 3. **Reinforcement for the well curb**

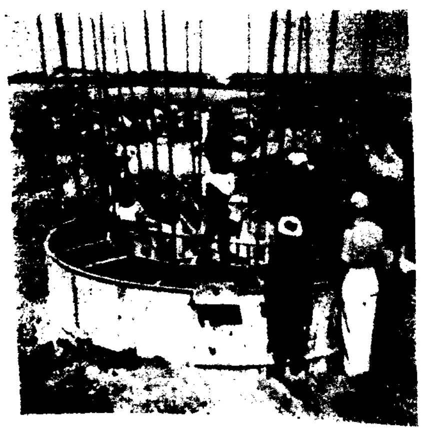


Photo 4.
Concreting the well curb

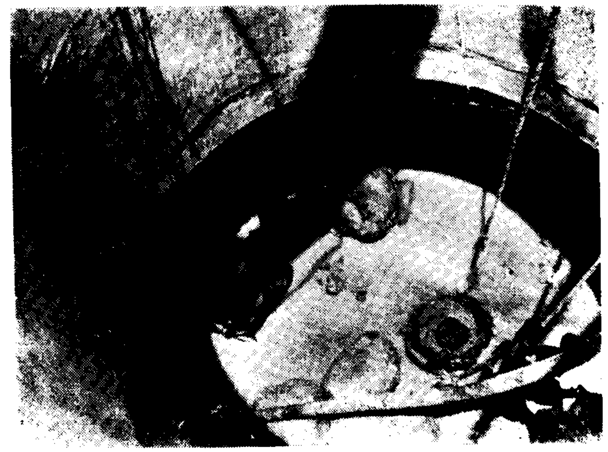


Photo 5. **Interior of a well with the earth under the curb cleared**

Photo 6. **Tremmie in use**

Photo 7. **Concreting the capping slab**

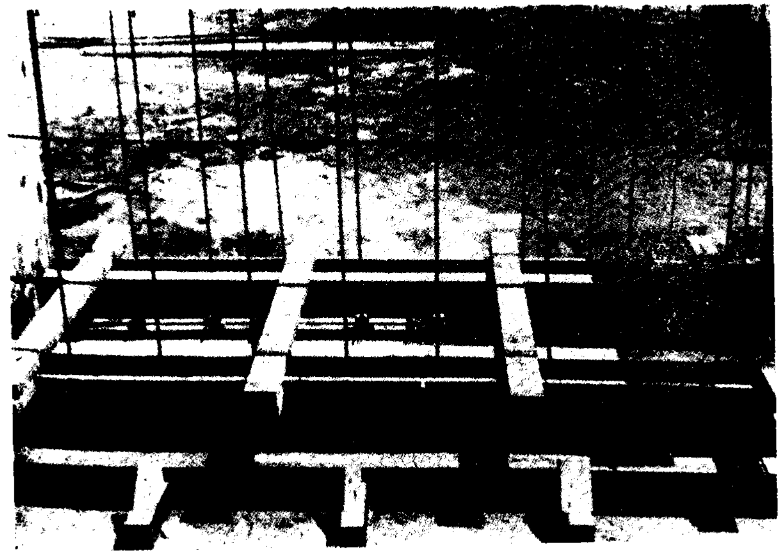


Photo 9. **Shuttering for the soffit slab**

Photo 8. **End block and diaphragms. Note the arrangement of ducts in the end block**

Photo 10.

Concreted beam

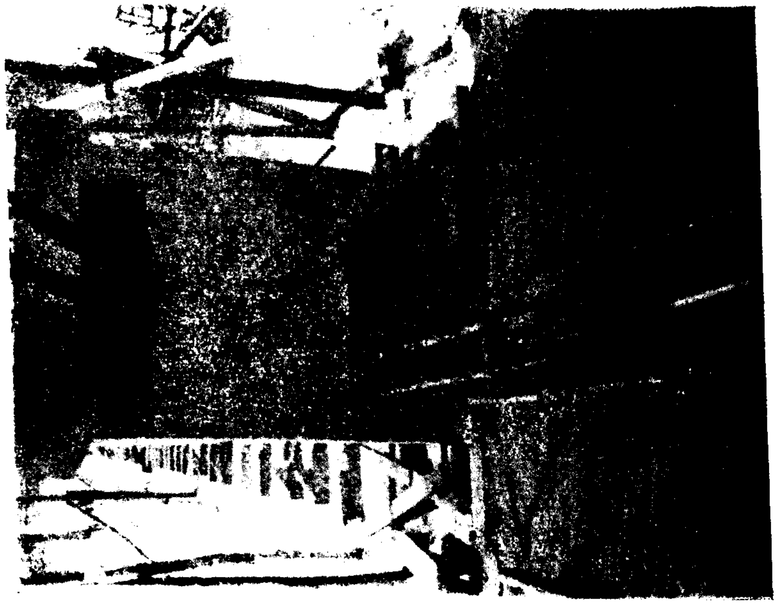


Photo 12. **Tensioning jack in position**

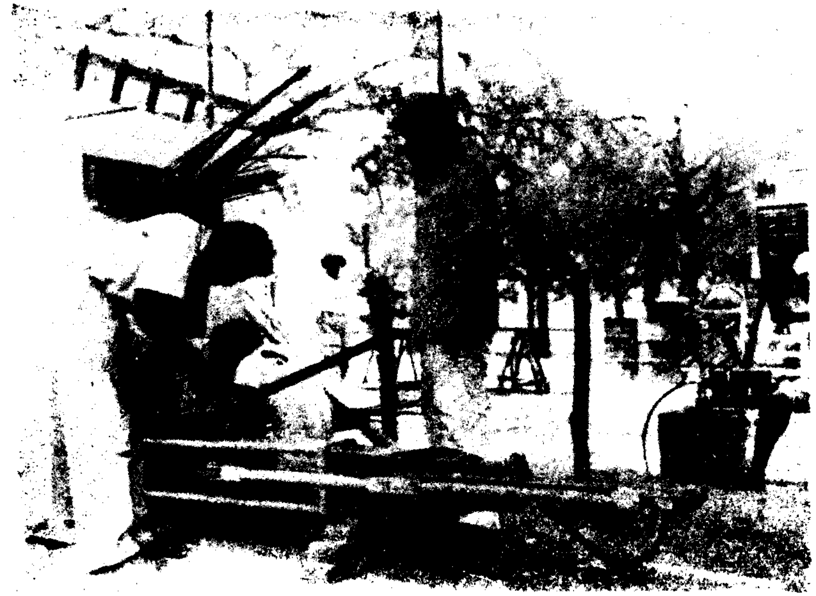


Photo 13. **Fixing of the wires after tensioning by driving in the wedge**

Photo 11. **Preparation of the cable**

Photo 14.

A beam being lifted for placing on a trolley

Photo 15.

Beam on sleeper crib

Photo 17.

Transfer of the front end of beam from the bank to the bed trolley

Photo 16.

Beam on trollies on the bank

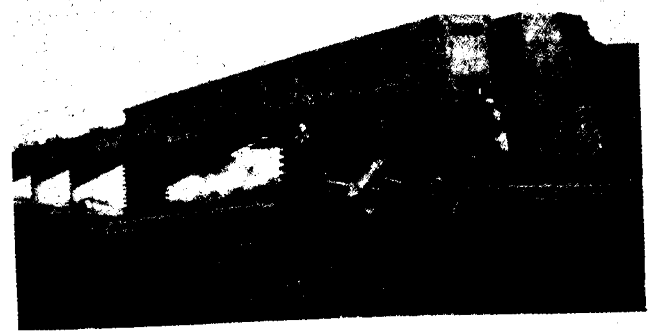


Photo 18. Transfer of the beam to the bed trollies

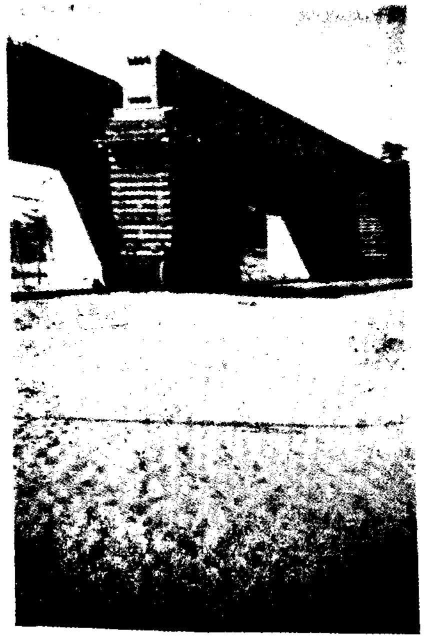


Photo 19.

A beam on the move on the river bed

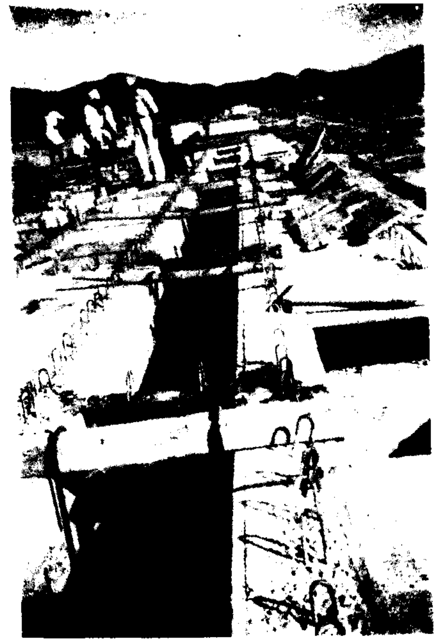


Photo 21.

Shuttering for the in situ diaphragm between the beams

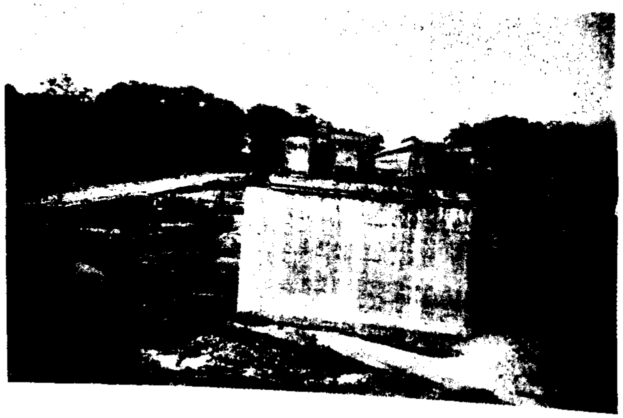


Photo 20. **Beams seated over a pier**

Photo 22.

Free end of the beam on rollers

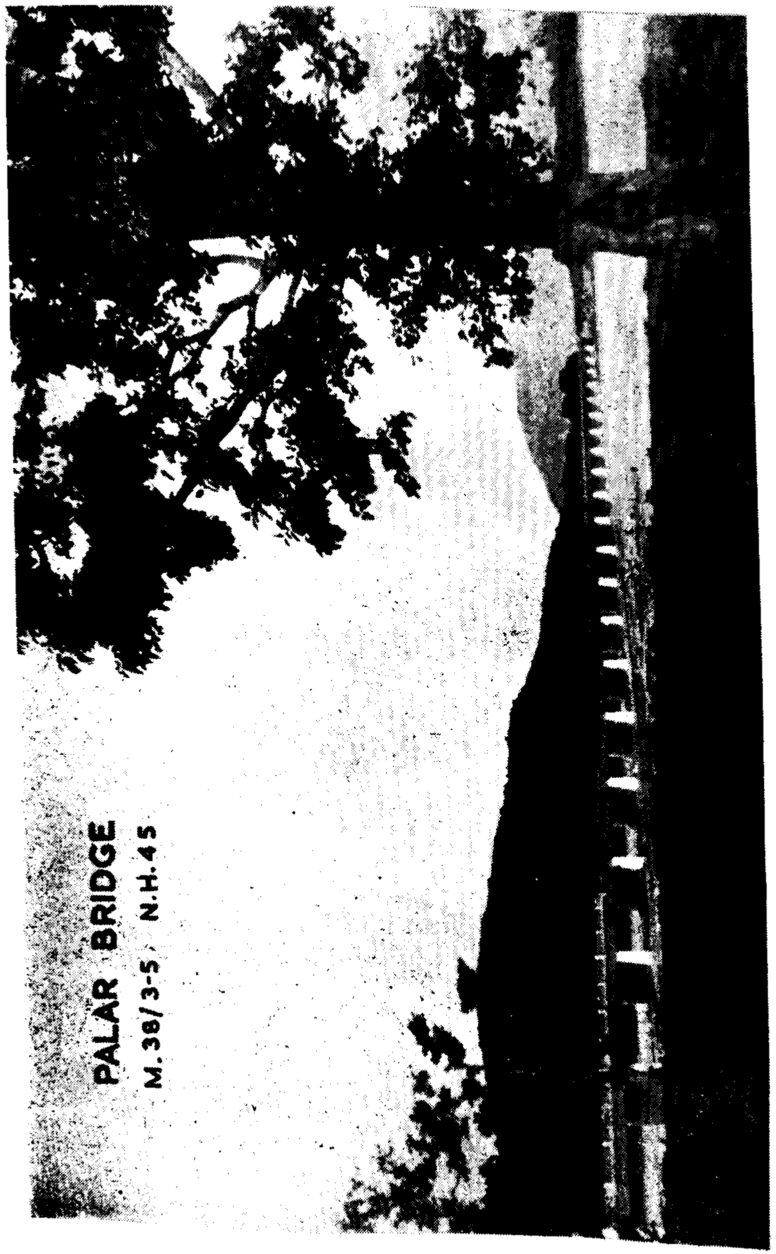


Photo 23

was sunk 7 ft below the calculated scour depth, it was felt that the well could rest on the boulder. As a precaution holes were drilled into the boulder and 12 dowel bars were put in before plugging the bottom. The extra pit that had been formed in the attempt to remove the boulder was also concreted.

In wells of pier 5, only sand was met with even after sinking 52.5 ft. As the calculated scour depth was about 27 ft above the bottom of wells as sunk, they were left in sand.

The progress on well sinking in one of the well is shown in Fig. 5.

7.4. Bottom Plug

After a well was sunk to required level, the bottom 5 ft of the well was plugged with concrete 1 : 2 : 4 with $1\frac{1}{2}$ in. gauge metal under water. The " Tremmie method " was adopted. The principle made use of in this method was that when a heavy liquid is poured into a lighter liquid the former sinks and settles down to the bottom. The " Tremmie " consisted of a hopper connected to a pipe 8 in. diameter. The pipe was taken down to the bottom of the well and the hopper was well above the level of water in the well. Photo 6

First the pipe was completely filled with sand. Concrete 1 : 2 : 4 mix with a consistency giving a slump of 6 in. was poured into the hopper. The hopper and the pipe, which were connected to a winch by means of wire rope and pulley were slowly raised by about 6 inches. The sand flows out a little and its place is taken by concrete of plastic consistency. In this way the whole pipe was filled with concrete. When the pipe was lifted at this stage by 6 in. and then lowered, some of the concrete in the pipe flowed out and the bottom of the pipe will be in concrete. Fresh concrete is then poured into the hopper and the pipe is lifted by 6 inches and lowered. This process is repeated. Care was taken to see that bottom of the pipe was always within the concrete that flowed out. A sinker was sent under helmet now and then to level up the concrete that had flown out of the pipe. Soundings were taken to ascertain the depth of concrete. This method gave very satisfactory results. Some of the wells were de-watered and concrete was found to be water-tight. A thin layer of slush was seen on the top of the plug. Usually it took only 5 hours to plug one well. The quantity of concrete involved was 326 cu. ft.

7.5. Sand Plug

Immediately after bottom plugging, sand plugging was started and completed. Sand plugging was done under water as sand will have minimum of voids when fully soaked in water. The top two ft of the wells were plugged with cement concrete of lean mix (1 : 4 : 8).

7.6. Capping Slab

Details of reinforcements of the capping slab are shown in Plate VIII. Steel shutterings were used. The concrete of 1 : 2 : 4 nominal mix with a water-cement ratio of 0.50 was used and compacted by vibration, Photo 7. Tests on six inch cubes taken on actual job showed a compressive strength of more than 4,000 lb per sq. in.

7.7. Piers

Plum concrete was used for piers. The proportion of concrete was 1 : 3 : 6 with 20 per cent plums. The pier was raised in four stages. Shutterings were all made of M. S. sheets 1/8 in. thick, stiffened with M. S. angles $1\frac{1}{2}$ in. × $1\frac{1}{2}$ in. × $1\frac{1}{2}$ in. While concreting each stage, 5/8 in. rods threaded at ends were fixed 1 ft below the top of shutters. For concreting the next stage, shutters were raised and fixed at bottom to the rods with bolts so that there was an overlap of one foot. At top, timber distance pieces were provided to maintain the required width. External timber struts were also provided to give rigid supports. Concreting was done in layers of 6 inches. About $5\frac{1}{2}$ gallons of water were used per sack of cement. Piers were provided at top with a R. C. distribution slab.

7.8. Abutments and Wings

Abutments and wings were also of plum concrete. In addition to the main wings, auxiliary wings were also provided to give an architectural effect.

8. PRECASTING AND PRESTRESSING OF THE BEAMS

The work comprised the following:

Work at costing yard,

Launching of precast girders, and

Cast in situ work,

8.1. Casting Yard

In order to take full advantage of precast work, the order of casting, prestressing, and release of the casting beds had to be pre-planned. It was planned to cast 8 beams per month and hence eight beds 4 on each side of a trolley line were prepared. The casting yard had an area of about 275 ft × 60 ft.

The casting bed consisted of a strip of concrete 41 in. wide and 1 ft deep and 94 ft 8 in. long (the length of the girders). A longitudinal camber of $1\frac{1}{4}$ in. was provided in the bed. The end 5 ft of the casting bed was given 2 ft deep concrete foundation. The edges of the bed were provided with channel 4 in. × 2 in. so that edges might not wear out. Casting and prestressing of girder was completed in the following five stages:

(a) Casting of end blocks,

(b) Casting of precast diaphragms,

(c) Casting of soffit slab,

(d) Casting of webs,

(e) Prestressing.

8.2. End Blocks

Reinforcement details of the end block are given in Plate IV. The end block bears a reaction of 500 tons and hence it is massive, heavily reinforced and solid but for the cable holes. Shutterings were made with M. S. plates 3/16 in. stiffened with channels 3 in. × $1\frac{1}{2}$ in. Steel vibrators plates with notches were welded on to shutters. Electrical form vibrators which have corresponding notches were fixed on to the shutters while concreting. Four form vibrators were used in addition to one internal vibrator.

For providing ducts in the end block, Magnel's standard rubber cores were used. These rubber cores are 1.97 in. × 1.97 in. with a hole of 13/16 in. for inserting a rod to keep the core straight. Two such cores, one on top of the other, were used. See Photo 8.

8.3. Diaphragms

There are 8 diaphragms between the end blocks, thus providing nine bays of 9 ft each. The shutters of the diaphragms were made of M. S. Plates stiffened with 3 in. × $1\frac{1}{2}$ in. × $\frac{1}{2}$ in. channels. For fixing electrical shutter vibrators, steel plates with grooves were provided. Vibrators were fixed to these plates by means of steel wedges. Correct position and shape of the holes for the cables were ensured by providing dealwood boxes made to shape and fixed to the shutters by bolts. After casting was over, these boxes were knocked out.

8.4. Soffit Slabs

The soffit slabs and 4-in. height of webs were cast in one stage. The shuttering consisted of timber framing for the sides. A pin vibrator was used, Photo 9.

8.5. Webs

It should be observed that there is a construction joint between the soffit slab and web. The web reinforcement provided is adequate for taking the principal tension created by prestressing. Hence it was considered safe to allow this construction joint in an horizontal plane. Like the shuttering for the end blocks and diaphragms the shuttering for the web was also manufactured using 3/16 in. mild steel plates stiffened by welding 3 in. $\times 1\frac{1}{2}$ in. $\times \frac{1}{2}$ in. channels at 2 ft centres. For maintaining correct width, of mortar blocks 5 in. $\times$ 2 in. $\times$ 2 in. with a hole inside were introduced between shutters and fixed with bolts which pass through the hole of the mortar block and shuttering. After concreting the web the bolts were taken out and the hole was filled up with mortar. For keeping the top of shuttering in position, tie rods 1 in. diameter which work in pipe sleeves were bolted. These shutters were very satisfactory.

8.6. Prestressing (Magnel Blaton System)

For each beam there were 8 cables of rectangular section with 32 wires in each. It was composed of layers of wires, 4 per layer. The wires were systematically arranged by using standard Magnel Blaton "grills". These grills were placed one at each diaphragm and end blocks. The wires maintain their geometric pattern throughout their length.

Magnel Blaton anchorages consist essentially of distribution plates, sandwich plates and wedges. The distribution plates are made of cast steel. They are fixed to end block by means of mortar after bush hammering. They distribute the cable force to the end block over such an area that the pressure on the concrete does not exceed 3,500 lb per sq. in.

Each anchorage consists of 4 sandwich plates. Each sandwich plate is capable of anchoring 8 wires. These sandwich plates bear against distribution plate.

The cable is first knit into grills, Photo 11 and, inserted in the girder through the cable holes provided in the end blocks and

diaphragms. Sandwich plates are then fixed against distribution plate by means of temporary clamps. Two wires pass into each slot of the sandwich plate.

Tensioning equipment consists of a hydraulic jack which tensions two wires at a time by means of a temporary grip, Photo 12. The Magnel Blaton hydraulic jack consists of a "strong back" to which two wires can be fixed by means of a steel master-wedge. The strong back is connected to a cross head by means of two steel links and the cross head itself is connected to the ram of the jack. The jack is supported on a frame and this bears against the sandwich plates. The jack is connected to a Tangye hydraulic pump having a pressure gauge. When the ram moves, the grip with master wedge also moves and pressure gauge on the pump records the load on the wire.

Next the actual elongation required at each pair during prestressing operation and the prestressing at the gauges are fixed. See Appendix 1.

A wire gauge is prepared for the required extension of 5–1/16 after an initial load of 1,000 lb. One end of the pair of wires to be stresssed is locked by inserting a wedge in the sandwich plates. From the stressing end the wire is tensioned such that the reading on the pressure gauge is 5,000. Now with a special tool, the wedge at the anchored end is driven hard and pressure gauge reading is noted. Driving of the wedge is stopped when the pressure gauge shows a constant reading when the wedge is being driven. After the wedge is driven hard at one end the tension in the wire is released such that the reading is 1,000 lb. At this stage the wire would be held just taut. The wire gauge is held against the sandwich plate, the initial mark is transferred to the wire that is being stressed with a coloured pencil. The pump is now worked to 10,000 lb which corresponds to design stress of 140,000 lb per sq. in. and the mechanical losses. These losses are (i) Pair loss, (ii) Jack and spring losses, (iii) Relaxation of wire during anchoring. After waiting for two minutes, wedges are inserted into the sandwich plate between the wires and driven hard by means of a special tool, Photo 13. The master wedge of the jack is now released. The wire gauge is held by the side of the stressed wire to see if the coloured pencil mark coincides with the gauge mark. Sometimes a relaxation of about 1/16 in. was observed.

Sometimes for the correct extensions the pressure gauge on the Tangye pump indicated a reading 250 lb - 500 lb on the high or low side of the correct reading. This was due to slight leakages in the pump and the jack brought about by worn out washers. Sometimes the spring of Bordon gauge required adjustments. Frequently the pressure gauge was tested in the field laboratory. For every 10 beams prestressed, one test was conducted on the high tensile wire for proof stress and Young's modulus. It is felt that it is much better to depend on measured extension than to depend on pressure gauge readings alone. The "hogging" of the beam was also measured after prestressing as this indicated whether beam had been over-prestressed or under-prestressed. On the whole the prestressing work was very satisfactory as very uniform results were obtained. Observations were recorded in the form given, Appendix 2. A sample posting of values is also given to afford a true picture.

8.7. Grouting

After prestressing was completed, the cable was completely encased in cement mortar grout. Wooden forms were used. Care was taken to see that air was expelled and grout entered the interstices between wires.

9. LAUNCHING

9.1. A method of launching the precast girders weighing nearly 60 tons each from the casting bed to the piers had to be evolved, taking into consideration local talent, materials and resources. Advantage was taken of the dry bed of the river to move the beams on a trolley line. The launching comprised of the following three stages :

(i) lifting the girders from the casting bed and placing on trolleys,

(ii) transfer of the girders from shore trolleys to bed trolleys, and

(iii) hauling of girders to the pier and their transfer from bed trolleys to the pier.

9.2. L-Plates 7/8 in. thick, stiffened at centre, are fixed to sides of the girder by means of 4 bolts $1\frac{1}{2}$ in. diameter. First, at one end two hydraulic jacks 35 ton capacity each are placed, the girder is lifted by about 6 in. and a sleeper is introduced. Similarly at the other end also a sleeper is introduced and girder made to rest on sleepers. Between the sleeper and the beam a ball rack is introduced at both ends. These ball racks consist of an ordinarily bull-headed rail placed with its web horizontal and steel balls laid on the web portion and covered over by an inverted channel. A screw jack which can work against the girder is fixed on the rail of the ball rack. By working these screw jacks the girders are moved sideways over the prepared track. See Photo 14.

9.3. At either end a crib of sleepers is built such that the girder rests on it with an overhang about 9 ft. Under this condition tension is not developed. Now the trolleys are introduced at both ends and the beam lowered on them by using hydraulic jacks. The girder is now hauled by means of a winch to the end of the bank. Photos 15 and 16.

9.4. A second set of trolleys is kept ready at the bed. These trolleys have tall cribs built on them so that when the girder is transferred on to them, the girder will be at the level of the pier top. Both the front trollies in the bed and the bank are placed in a line and the girder is cantilevered out by building a crib about 10 ft from front end of the girder. The ball rack is now introduced underneath the girder and by working the winch, one end of the girder is moved on to the front bed trolley and lowered on it. The temporary crib that was built on shore is now removed and front bed trolley is pulled by means of a winch. The hind end of the girder is transferred on to hind bed trolley by adopting a procedure similar to that adopted for transferring front end of girder. Photo 17 and 18.

9.5. The girder is now hauled by means of a relay of winches kept at every 500 ft. When the girder comes against the required span, a rail nest is placed between the cribs of the trolleys and cut water of pier. A ball rack is introduced beneath the girders and girder is moved over the rail nest on to the piers by means of screw jack. Photos 19 and 20.

9.6. The cost of moving the girder worked out on an average to only Rs 3 per ton.

10. IN SITU WORK

In situ work consists of casting diaphragms between the beams and laying the deck slab.

10.1. Diaphragm Between Beams

Steel shutterings were used for casting the diaphragms. The bottom of the shuttering was made of a channel and this was hung from the main reinforcement itself by means of J-bolts. After removal of shutters the bolts were cut, caulked in and grouted so that there was no risk of setting in, compaction of concrete was done by using poking rods. Pin vibrators could not be used because of the thin section, Photo 21.

10.2. Deck Slab

The shuttering for deck slab consisted of mortar slab 1 in. thick. These were first used to cover the top of the beam. They proved very successful and later this was used between girders as well. The mortar slabs were reinforced with waste high tensile wire bits. The slabs were placed in position by providing 1 in. × 1 in. grooves in the girder. The slabs were left in place even after concreting.

For compacting concrete of deck slabs both pin vibrators and screed vibrators were used. Water-cement ratio was 0.5. The strength attained on 6-in. cubes was uniformly more than 4,000 lb per sq. in.

11. BEARINGS

11.1. The fixed bearing of the girders consists of a mortar pad 1 in. thick with a dowel pin connecting the beam with the distribution slab over pier. Girders are lowered on a wooden frame of required dimensions. $1\frac{1}{4}$ in. dowel pins are dropped in position through the holes provided in the girder and the distribution slab. Grout of cement mortar 1 : 2 is poured through one of the holes and worked up with a tamping rod. The grout fills up the hole in the distribution slab and the timber frame and rises into the second hole. Timber frame is left in position even after the mortar had set.

11.2. The bearing at free end consisted of a simple roller of mild steel, Plate IX. The top plates are fixed to the girder bottom while casting. The rocker is fixed in position. The lowering of beams did not present any special difficulty.

12. WEARING COAT AND HAND RAILS

12.1. Wearing Coat

The wearing coat is of $1 : 1\frac{1}{2} : 3$ concrete with 1 in 132 cross camber. The expansion joints are placed at 192 ft from each other and construction joints at 48 ft. The top of the slab is painted with bitumen before laying the wearing coat.

12.2. Hand Rails

Posts and pilasters are cast in situ while rails and panels will be precast units.

12.3. Pylons

Attractive pylons are proposed at the ends of the bridge to symbolize the technical excellence incorporated in the structure.

13. FIELD LABORATORY

As the work was the first of its type carried out by the department, close supervision was necessary. So a field laboratory was provided for carrying out routine tests. Soils that were met with during well sinking were also sampled out.

Among the regular tests that were conducted were the cube strengths on concretes of various mixes, analysis of aggregates and bulk tests on sand. Field observations were conducted on prestressing 'hog' and temperature expansion of girder and deflections.

The machinery used on the construction is given in Appendix 3.

14. COSTS

The total estimated cost of work is Rs 25 lakhs. Cost of foundations is Rs 5.9 lakhs. Cost of substructure is Rs 3.2 lakhs. Cost of superstructure alone is Rs 10.5 lakhs. The cost per running foot of bridge works out to Rs 1,051.

ACKNOWLEDGMENT

The Author acknowledges the help given by Sri K. Srinivasa Rao, the Special Assistant Engineer, in getting out the details for this Paper.

APPENDIX 1

CALCULATION OF EXTENSION FOR THE PRESTRESS

Initial prestress applied: 1,40,000 lb per sq. in.

Losses after anchoring:
- (i) Shrinkage of concrete
- (ii) Creep of concrete
- (iii) Relaxation of steel.

An allowance of 15 per cent has been made in the design calculations to cover all these losses.

Losses due to prestressing
- (i) Slip at anchorage. $\frac{1}{16}$ in. allowed per wedge.
- (ii) Pairs loss.

At transfer maximum compression in concrete: 1795 lb per sq. in.

At transfer compression in concrete at ends: 1175 lb per sq. in.

Therefore, average compression (Parabolic variation) = $1175 + 2/3 \times 620$

$$= 1588 \text{ lb per sq. in.}$$

Average strain in concrete $= \dfrac{1588}{E_c}$

Strain lost in first pair of wires $= \dfrac{127}{128} \times \dfrac{1588}{E_c}$

Strain in last pair $= 0.$

Therefore, average strain lost $= \dfrac{127}{2 \times 128} \times \dfrac{1588}{E_c}$

Average stress lost $= \dfrac{127}{2 \times 128} \times \dfrac{1588}{E_c} \times E_s$

$$= \frac{127}{2 \times 128} \times 1588 \times 7$$

$$= 5{,}500 \text{ lb}$$

Pairs Loss $= \dfrac{5500}{140{,}000} \times 100$

= 4 per

Length of wire =	Overall length	94 ft 8 in.
	Distribution plates	0 ft 4 in.
	Sandwich plates	0 ft 4 in.
	Gauge mark	0 ft 4 in.
		95 ft 8 in.
		= 1148 in.

Strain ratio for wire at 1,40,000 lb per sq. in.

$$E_s = 30.5 \times 10^6$$

Initial gauge reading before marking a mark on wire:

$$= 1{,}000 \text{ lb}$$

Spring losses for this load $= 1000 - 250$ lb

$$= 750 \text{ lb}$$

Stress on pair of 0.200 in. diameter wire $= \dfrac{750}{0.0628}$

$$= 12{,}000 \text{ lb per sq. in.}$$

Additional stress to be applied = 1,28,000 lb

Elastic strain $= \dfrac{128000}{30.5 \times 10^6} \times 1148 = 4.01$

Pairs loss 4 per cent	= 0.15
Slip 1/16 in.	= 0.06
	5.06

or say $5\frac{1}{16}$ in.

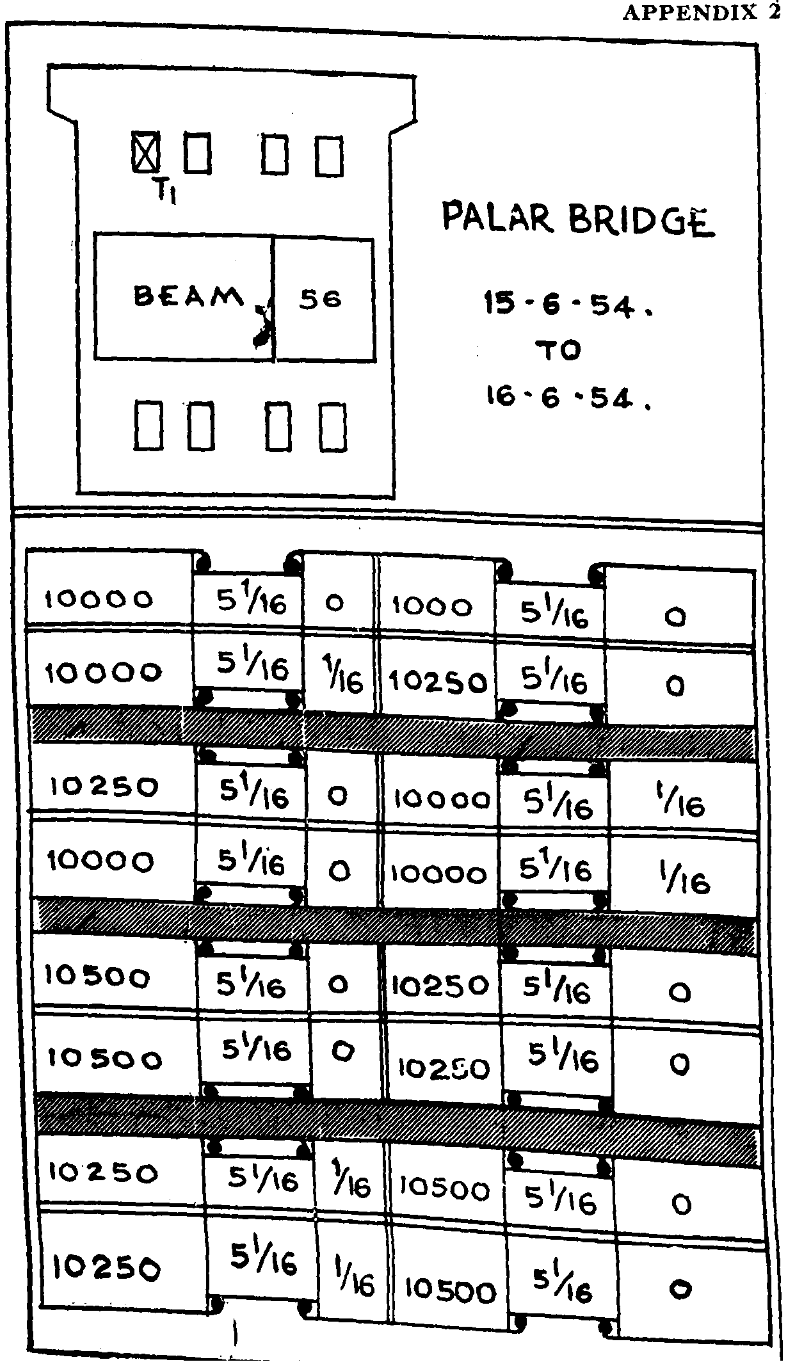
Pressure gauge reading:

Load on a pair of wires = 1,40,000 × 0.0628	= 8,800
Allow for 5 per cent jack losses	= 440
Allow for springs extended	= 400
Slip and pair losses	= 440
	10,080

say 10,000 lb

Therefore minimum gauge reading is 10,000 lb for applied extension of 5-1/16 in. after an initial load of 1,000 lb.

APPENDIX 2



APPENDIX 3

PLANT AND MACHINERY USED

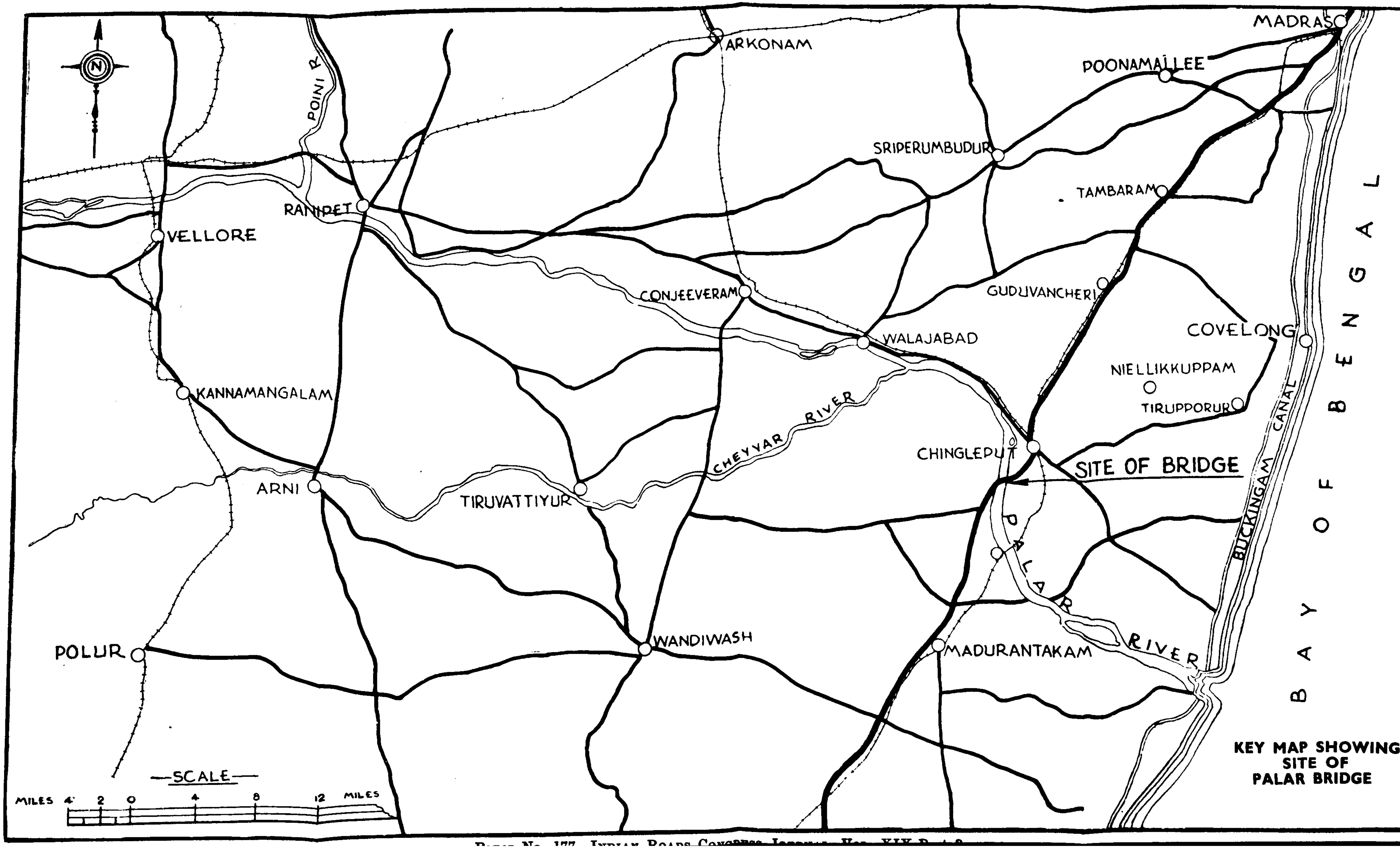
Substructure and Foundations

Atlas Diesel Air Compressors	... 2
Centrifugal Pumps 6 in.	... 1
Centrifugal Pumps 5 in.	... 1
Petter Engine Pump set 3 in.	... 2
Sump Pumps	... 2
Diving Helmets	... 11
Concrete Mixers 5-7½ cu.ft. capacity	... 2
Winches 2 ton capacity	... 4

Superstructure

Liner Cumflow Mixer 5 to 7½ cu. ft.	... 1
Concrete Mixers 5 to 7½ cu. ft.	... 3
Electric Generator Plant	... 1
Electric Welding set	... 1
Form vibrators 230 volts	... 12
Electrically operated Pin Vibrator	... 1
Flex-shaft Pin Vibrators	... 4
Lifting Jacks coupled to Tangye Hydraulic Pumps 35 tons, 6 in.	... 4
Lifting Jacks 16 tons 10 in. rise	... 4
Lifting Jacks 24 tons 10 in. rise	... 2
Winches 10 tons	... 3

PLATE I



KEY MAP SHOWING SITE OF PALAR BRIDGE

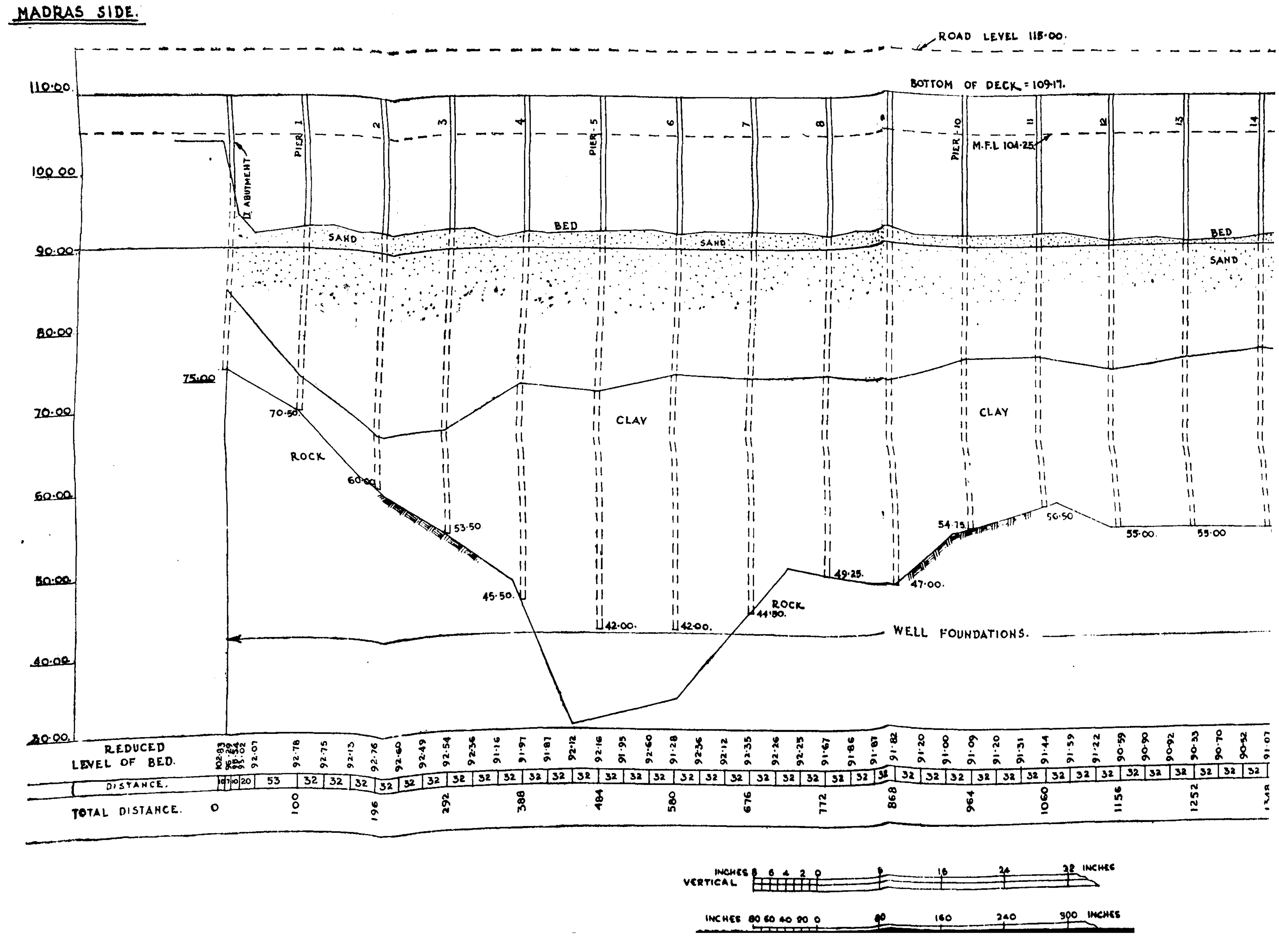
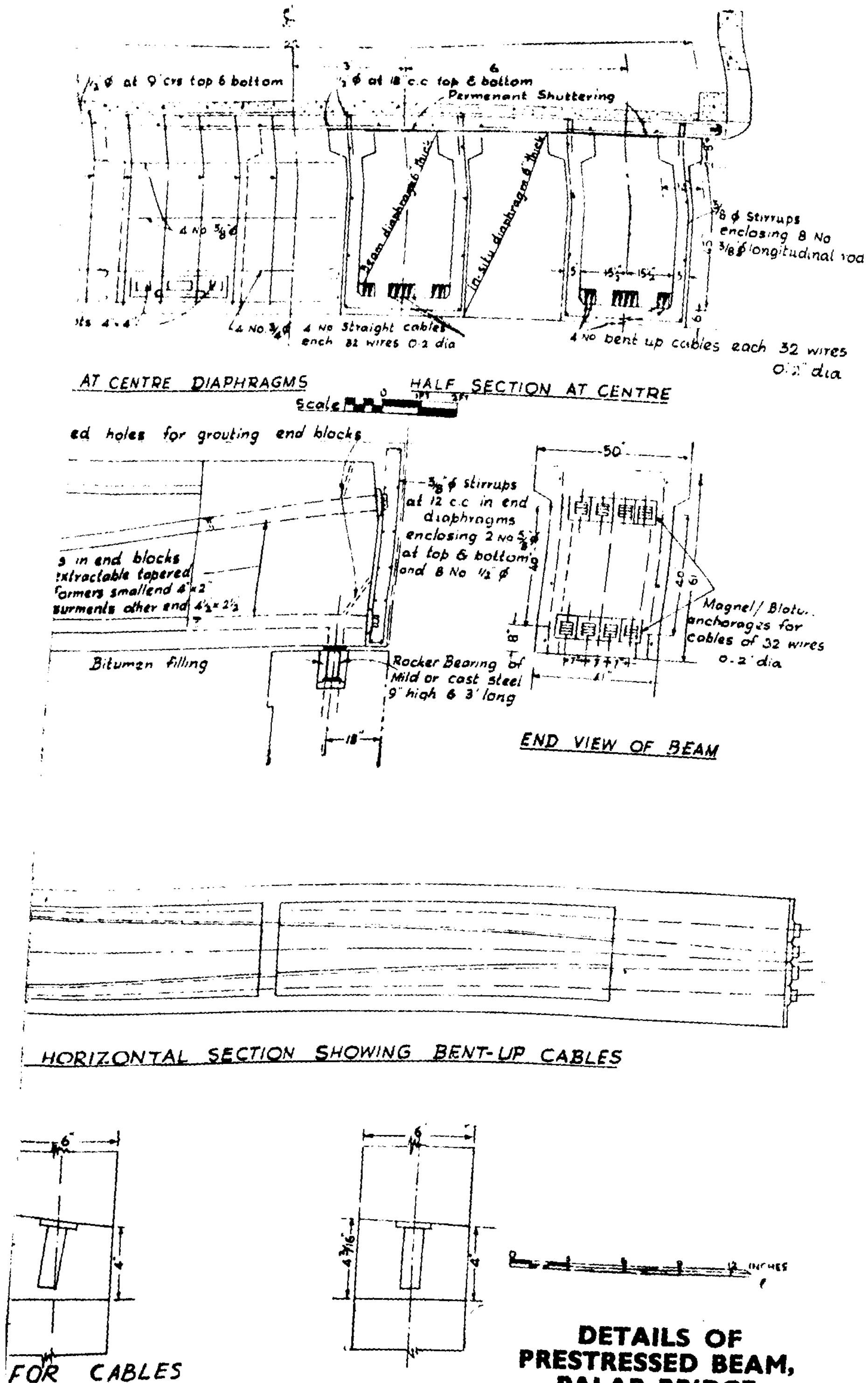
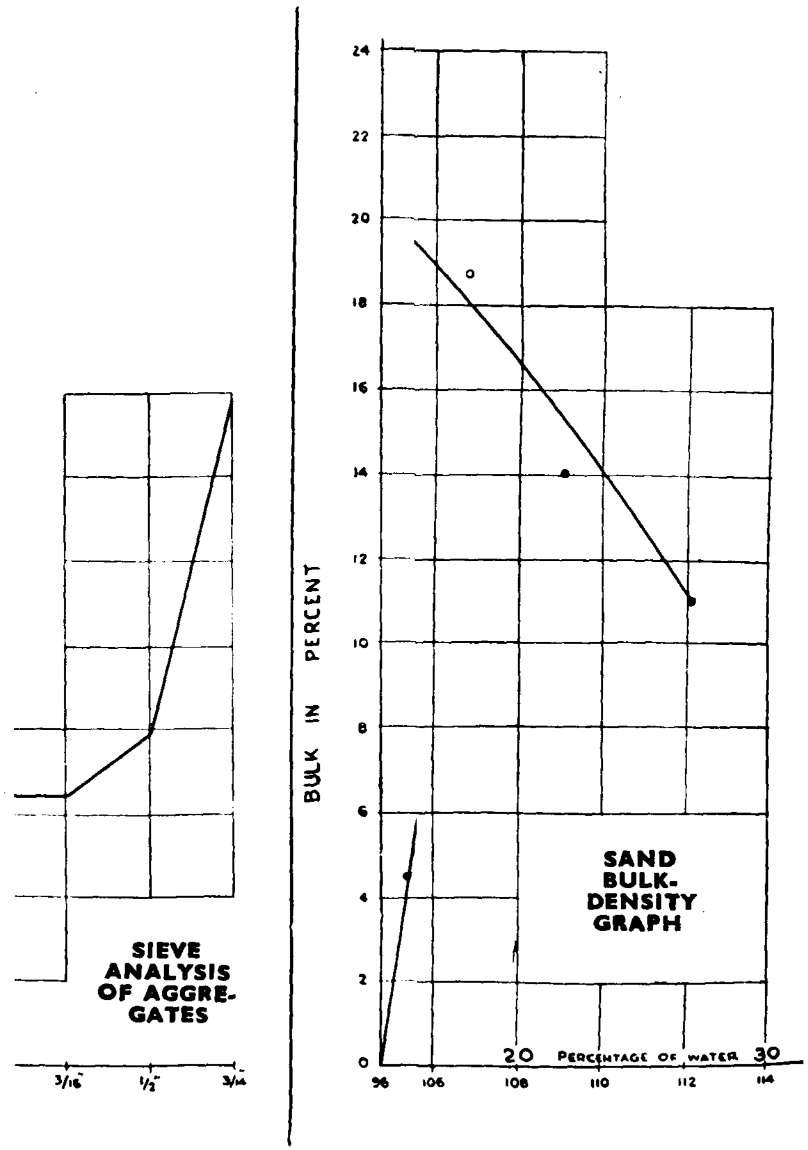

MADRAS SIDE.
ROAD LEVEL 115·00.
BOTTOM OF DECK = 109·17.
M.F.L 104·25.
ABUTMENT
PIER 1
PIER 5
PIER 10
BED
SAND
CLAY
ROCK
WELL FOUNDATIONS.
110·00
100·00
90·00
80·00
75·00
70·00
60·00
50·00
40·00
30·00
70·50
60·00
53·50
45·50
42·00
42·00
44·80
49·25
47·00
54·75
56·50
55·00
55·00
REDUCED LEVEL OF BED.
DISTANCE.
TOTAL DISTANCE.
0
100
196
292
388
484
580
676
772
868
964
1060
1156
1252
VERTICAL
INCHES 8 6 4 2 0 8 16 24 32 INCHES
INCHES 80 60 40 20 0 80 160 240 300 INCHES

PLATE IV



DETAILS OF PRESTRESSED BEAM, PALAR BRIDGE

PLATE V



SIEVE ANALYSIS OF AGGREGATES
3/16"
1/2"
3/4
BULK IN PERCENT
SAND BULK-DENSITY GRAPH
PERCENTAGE OF WATER
20
30
0
2
4
6
8
10
12
14
16
18
20
22
24
96
106
108
110
112
114

PANEL DISCUSSION

XIX SESSION OF THE INDIAN ROADS CONGRESS

The milages of water-bound macadam and black top roads in India are given below:

Year ended 31st March	Milage of W. B. M.	Milage Black topped
1947	77,575	10,285
1948	79,089	10,796
1949	80,062	11,275
1950	82,796	12,132
1951	83,488	12,588

This shows that the milage of black topped roads is increasing at a rapid rate, and it is likely that the rate of increase will be higher still during the Second Five-Year Plan period.

Nearly 90 per cent of the black top roads are surface dressed. In view of the importance of this method of surfacing, the questionnaire given below has been prepared to elicit information on the practices obtaining in various States and pool the experience of various engineers. It is the intention to use the information so collected in framing a standard specification on the subject.

The questions will be discussed at the General Session of the Roads Congress to be held at Srinagar in June 1955. Members who are enable to attend the Session, or to take part in the discussion, may forward their replies to the questions to the Secretary, Indian Roads Congress.

QUESTIONNAIRE ON SURFACE DRESSING WITH BITUMINOUS BINDERS

1. **General**

1.1. State the physical or other conditions which prompted you to the use of surface dressing as against other specifications of black topping on an existing surface of:

a. Water-bound macadam with

1. Stone metal
2. Brick
3. Kankar

b. Moorum

c. Laterite

d. Concrete

1.2. In general what are the limits of traffic intensity, for which surface dressing on roads mentioned in 1.1 has been found to be successful? Give details of

a. Iron tyred traffic

b. Pneumatic tyred traffic

c. Combined traffic

1.3. In your experience how far does surface dressing improve the riding qualities of a corrugated water-bound macadam road? To what extent is the beneficial effect, if any, related to the type of binder?

1.4. What is your experience of the life and behaviour of surface dressing in plain and hilly country?

2. Design

2.1. Do you generally apply "one-coat" or "two-coat" surface dressing?

2.2. When do you use one-coat surface dressing and for what intensities of traffic?

2.3. When do you adopt two-coat surface dressing? Give easons and intensities of traffic.

2.4. State the basis on which you fix the quantity of a binder required for surface dressing (i) one-coat, and (ii) two-coat?

2.5. Do you consider that the quantity of binder depends on the following factors?

(a) Traffic intensity

(b) Types of surface receiving the surface dressing

(c) Qualities of the binder itself (Asphalt Emulsion, Tar, etc.)

(d) Quantities and qualities of chips

(e) Maximum and minimum temperature of the place

(f) Rainfall and climate of the place

(g) Boiler and spraying pump or hand pouring.

State how the quantity you used was influenced by each one of the above factors.

2.6. State the basis on which you fix the quantity of chips required for surface dressing in (i) one-coat work, and (ii) two-coat work?

2.7. Are light chipping carpets used in your area? If so, do you find any difference, in service behaviour between a ¾ in. thick, pre-coated chipping surface and a 2-coat surface dressing.

2.8. Give the specifications you use for the pre-coated chipping carpet.

2.9. Is the ¾ in. thick pre-coated chipping carpet more economical than 2-coat surface dressing? If so, how?

2.10. In 2-coat work do you use the same binder or different kinds of binder for the first and the second coat? If so, why, and with what results?

2.11. Do you take the gradient into consideration before deciding on the details of the black topping specification for hill roads?

3. Priming

3.1. Do you find any marked difference between a road where primer was used, and that where primer was not used, before the application of surface dressing?

3.2. How much primer per hundred sq. ft. do you generally use?

3.3. Do you use different types and quantities of primer for different types of receiving surfaces? If so, what are the variations?

4. Chips

4.1. What sizes of chips do you use for the 1st coat and the 2nd coat and what methods do you adopt to secure the specified sizes?

4.2. Do you use chips of a uniform size? If so, why do you prefer these to graded chips?

4.3. Do you use cubic chips? If so, do they withstand traffic better than elongated or rounded chips? Do cubic chips adhere better to the binder and are, therefore, swept off the road surface?

4.4. What quantity of chips do you use per hundred sq. ft. for surface dressing (i) one-coat (ii) two-coat?

4.5. Do you use lower grades of aggregate like brick chips, coarse sand, laterite or kankar screening and pea-gravel etc. for surface dressing? How do they compare with higher grades of aggregate in service behaviour and life.

4.6. Describe the kinds of stone (petrographic classification) that you have found unsuitable for surface dressing. Give reasons and the kind of failure.

4.7. Do you use graded chips? If so, do you use the bigger sizes in the first coat and the finer ones in the second coat? What is the effect of this practice on the riding qualities and life of the road?

5. Binder

5.1. What are the different types and grades of binder you use for surface dressing?

5.2. Do you prefer hot or cold cut-backs to straight run bitumen? If so, why?

5.3. Do you use emulsion for surface dressing? If so, why do you prefer it to straight run bitumen and cut-backs?

5.4. Have you used tar-bitumen mixtures or any other mixtures such as rubber powder and bitumen?

6. Construction

6.1. Which season do you consider suitable for surface dressing work?

6.2. What precautions do you take when you have to lay a surface coat out of the proper season?

6.3. How long do you allow a newly constructed W. B. M. surface to set under traffic before the application of surface dressing?

6.4. Do you observe "sweeping off" of chips from the newly applied surface dressing? If so, what methods do you adopt to prevent this?

6.5. Do you edge surface dressing work? Do you find any difference in behaviour of a surface dressing with and without edging?

6.6. Which type and weight of roller do you use for surface dressing? Do you find any difference in behaviour, or the texture of the finished work, owing to the weight of the roller?

6.7. Have you done surface dressing work soon after the rains when the ground was wet? Do you notice any difference between stretches done when the ground was wet and dry?

6.8. What precautions have you taken to prevent bleeding of surface dressed carriageways, and, if the normal quantity of binder was used according to your general practice, what do you think was the cause of bleeding?

6.9. How do you normally apply binder, by spraying machine or hand pouring? If hand pouring is adopted, do you prefer to use brushes or rubber squeegees?

6.10. Do you advocate the progressive build up of a carpet of one inch or more in thickness by successive applications of surface dressing?

6.11. Under what conditions do you consider it necessary to scrape off the old surface dressing before putting on a renewal coat? If you do not remove the old surface, how do you ensure proper bonding?

Paper No. 178

TRAFFIC ROTARIES

By

THE SPECIFICATIONS AND STANDARDS COMMITTEE OF THE INDIAN ROADS CONGRESS

[Following precedent, the recommendations of the Specifications and Standards Committee on Traffic Rotaries are published in the Journal for discussion and comments of Members before finalization as approved Standards of the Congress.]

CONTENTS

SYNOPSIS

This Paper discusses the various factors that influence the design of traffic rotaries and lays down standards for:

(i) the radius of the central island,

(ii) the minimum and maximum weaving lengths and weaving angles,

(iii) the width of the rotary roadway,

(iv) the radii of entry and exit curves, and

(v) the pavement widths at entrance and exit.

The influence of the slow-moving bullock cart on the capacity of the rotary is discussed and, in the absence of observed data, tentative recommendations for evaluation of the slow-moving vehicles in terms of passenger cars are made.

Desirable practice in respect of superelevation and camber at the rotaries is given. The construction of channelising islands, and special considerations for footpaths, cycle tracks, and illumination are detailed.

The limitations, advantages and disadvantages of rotaries are mentioned.

Detailed designs of some rotaries constructed in Delhi are included.

1. INTRODUCTION

1.1. When vehicles negotiate an intersection they may conflict with one another. The design of an intersection should provide for the flow of traffic in a manner that would prevent or resolve the conflict, obviate the inherent traffic hazards, and reduce the wear and tear of vehicles. Conflict at road crossings at the same level can be reduced or avoided by one of the following three ways of control:

(1) signals manually operated by the police;

(2) automatic signals which are actuated by traffic or have fixed cycles; and

(3) construction of a properly designed intersection. One of the many methods of dealing with the problem is the installation of a traffic rotary.

1.2. Statistics show that a rotary is freer from accidents than crossings controlled by signals. Also, tests have shown that a single stop and re-start at a traffic signal by a car travelling at 35 m.p.h., could consume fuel required for travelling 300 yards at the same speed without check. In a test[23], a heavy vehicle, fitted with a 125 H. P. diesel-engine, was doing 12 miles per gallon when running steadily at 14 m.p.h., but stopping and restarting once a mile reduced the performance to a little over 10 miles per gallon. Tests on two arterial roads on the outskirts of London—one having 5 rotaries in a distance of 4½ miles, and the other having 10 signal installations in a distance of 5 miles revealed that the test car did 38 miles to the gallon on the first and 24 miles to the gallon on the second road.

1.3. Therefore, while the installation of traffic signals is the only effective method of control where the junction is hemmed in by expensive property, a rotary is preferable where sufficient land is available.

2. WHAT IS A 'TRAFFIC ROTARY'

2.1. A traffic rotary may be described as an enlarged road intersection where all converging vehicles are forced to move round an island in one direction before they can "weave" out of the traffic flow into their respective directions radiating from the island. The central island may be of any shape permitted by site conditions.

2.2. The objects of providing a rotary are to reduce the areas of conflict, control the angles of approach, and eliminate the necessity of stopping.

3. CONDITIONS JUSTIFYING THE CONSTRUCTION OF A TRAFFIC ROTARY

3.1. The American Association of State Highway Officials, in their "Policy on Rotary Intersection" suggest that the lowest limit of traffic volume for which a rotary may be justified is about 500 vehicles per hour on all the intersecting roads combined, and the maximum limit beyond which the rotary may not function efficiently is placed at 5,000 vehicles per hour. If, however, a large proportion of the traffic is "turning traffic", the use of a rotary outside the above limits might be justified. A rotary is also justified when the through traffic on a road at an intersection is obstructed or delayed by cross traffic, the latter amounting to about 50 per cent of the total traffic.

3.2. The Ministry of War Transport, U.K., have laid down that there is a *prima facie* case for the construction of a rotary at the crossing of a light trafficked and a heavily trafficked road at which the volume of traffic entering from the minor road is more than 25 per cent of all the traffic entering the intersection, or where many vehicles make a right-hand turn.

3.3. The behaviour of traffic in India is greatly influenced by the slow-moving, animal-drawn vehicles. In some urban areas, however, motor traffic is predominant. Generally, therefore, in India traffic rotaries are necessary at present at important road junctions in built-up areas. Further, the conditions imposed by mixed traffic have an adverse influence on the average traffic speed. In order that intersections may not cause a further reduction in the average speed, **traffic rotaries are recommended where the intersecting motor traffic is about 50 per cent of the total motor traffic on all the intersecting roads or where the fast traffic turning right forms at least 30 per cent of the total.**

4. ADVANTAGES

4.1. The advantages of traffic rotaries are :

(a) Consistent journey times are easily obtained by subordinating each converging traffic stream in favour of traffic as a whole.

(b) Traffic turning right has equal facilities and is subject to no greater delay than traffic turning left or that proceeding straight through to the opposite side of the rotary.

(c) Rotaries can be constructed for the junction of several converging roads with a probable limit of seven roads.

(d) Rotaries can accommodate total traffic up to 5,000 vehicles per hour, and enable radial streets to carry traffic to their full capacity.

(e) It is the simplest of all controls.

(f) Maintenance costs are less than for control by traffic signals.

(g) Vehicle operation costs are less because stopping and starting are avoided, and the vehicle is kept moving. More comfortable travelling is ensured.

(h) A check on the speed of vehicles is automatically enforced and serious collisions are avoided.

(i) Supervision of traffic movement is not required even when turning movements are heavy.

(j) The continuous and steady flow enables public service vehicle to maintain proper spacing instead of becoming bunched together and crowding bus stops, as often happens at signal-operated intersections.

5. LIMITATIONS

5.1. Traffic rotaries are not suitable :

(1) where space is limited and costly ; a rotary requires a comparatively large area of land which pushes up the cost in built-up areas ;

(2) where pedestrian traffic is large, a traffic rotary by itself is not sufficient to control traffic and has to be supplemented by traffic police ;

(3) where the distance between intersections tends to be small, rotaries become ineffective and troublesome :

(4) where the angle of intersection of two roads is very acute or if there are more than seven intersecting roads.

5.2. In places where motor vehicles, cyclists, and pedestrians have to be provided for separately, the design of a rotary becomes elaborate and requires a very large area. Before adopting a rotary for a particular case, these points should be carefully considered.

6. BASIC REQUIREMENTS OF INTERSECTIONS

6.1. The point where the possible paths of two vehicles ntersect is called a point of conflict. The area containing all ossible points of conflict is called the area of conflict. An rea in which a vehicle is subject to conflict by more than one ehicle at the same time and place is termed a major conflict rea ; but when the vehicle is subject to conflict by one vehicle nly it is termed a minor conflict area. Fig. 1 shows the areas of onflict for an intersection of two roads at right angles to each ther.

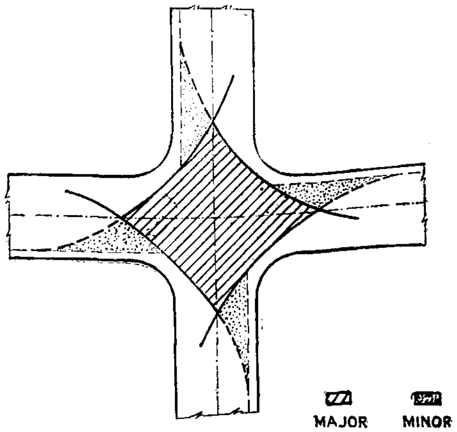


Fig. 1. Areas of traffic conflict when two roads intersect at right angles

6.2. The principles underlying intersection design are :

(1) The area of conflict should be as small as possible and for this the angle of approach of the vehicles should be as large as possible, and a channelising island should be provided.

(2) When two vehicles approach each other at an angle, the relative speed of the two vehicles increases with their angles of approach.

(3) As the relative speed increases, the judgment of the drivers in respect of time and distance is likely to become more and more inaccurate, leading to a greater and greater liability of accidents.

(4) Adequate mutual visibility should be provided for the vehicles approaching the intersection. There is greater mutual visibility at wide angles of approach than at acute angles of approach.

(5) Sudden changes of path should be avoided.

(6) Adequate provision of appropriate geometric features such as turning radii, width of carriageway, and weaving length should be made.

(7) Appropriate signs to warn drivers of the existence of the intersection and good illumination at night are necessary for efficient functioning.

(8) Adequate provision should be made for the safe passage of pedestrians and cyclists, if their numbers are large.

7. DESIGN SPEED

7.1. In the design of a rotary the speed for which it has to be designed is an important consideration. In the I.R.C. Paper No. 119 on " Horizontal and Transition Curves for Highways ", the Specifications and Standards Committee recommended the following design speed :

Class of Road	Design speed on the straight in flat or rolling country (m.p.h.)
National & State Highways	50
Major District Roads	40
Other District Roads	30
Village Roads	20

7.2. The speed of motor vehicles in built-up areas is always less than that on straight reaches in open areas. Observations in America have shown that the speed attained in urban areas is generally about 0.7 of that attained in rural areas.

7.3. Although similar observations have not been made in India, the ratio of urban speed to rural speed in India would be lower than the American ratio because of the heterogenous character of traffic, a large proportion of which consists of slow moving animal-drawn vehicles. The main roads, however, will be used largely by motor vehicles and, therefore, it would be reasonable to take the speeds in urban areas as only a little less than 0.7 of the speeds in rural area, e.g.

Class of Road	Recommended design speed in the straight in urban areas (m.p.h.)
National Highways	30
Major District Roads	25
Other District Roads	20

7.4. **Relation between Speed at Turnings and Speed on the Straight.**

A driver on approaching a turn or an intersection, reduces his speed :

(i) to appreciate the amount and degree of turning involved ;

(ii) to adjust his transition ;

(iii) to meet any unexpected situation caused by some defect in the geometrical design of the turn, or unevenness of road surface ; and

(iv) to avoid conflict with any other vehicle.

Observations in America show that this reduction in speed near an intersection is about 30 per cent of the original speed. As rotaries have to be consciously designed for traffic safety, it would be proper to assume that the above ratio holds good under Indian conditions also. On this basis, corresponding to speeds of 30, 25 and 20 m.p.h. on the straight, the speeds at intersections would be 21, 17.5 or 14 m.p.h. respectively.

It would not be correct to design rotaries for very low speeds as that would not be conducive to economy in the time of travel which is one of the advantages of rotaries compared to other kinds of control. Nor would it be desirable to adopt very high speeds, as the higher the design speed the bigger the rotary has to be and that would lead to high cost and increase in the time taken to travel round the rotary. Also, since in India both the slow-moving, animal-drawn traffic and motor vehicles use the road and as rotaries are generally justifiable only in urban or semi-urban areas, **they should be designed for speeds of 25 and 20 m.p.h. The design speed of 25 m.p.h. should be adopted when one (or more) of the converging roads is (are) important. In all other cases, a speed of 20 m.p.h. should be adopted.**

8. THE CENTRAL ISLAND

8.1. The central island is most efficient when its outline is composed of a series of curves of large radii, avoiding corners. Sharp corners are handicaps to traffic flow and lead to the formation of unused areas. The outline of the island may be made symmetrical but sometimes such a design may hamper flow of traffic on some portions of the rotary.

8.2. The actual shape of the island will depend on the number and disposition of the intersecting roads. The exact shape is determined by connecting the sections of the road round the central island to form a closed figure giving at least the minimum weaving length between adjacent radial roads. The whole must then be adjusted to suit the curvature corresponding to the assumed design speed.

8.3. The central island should be so placed as to interrupt all the intersecting roads, thus forcing a marked left turn and preventing too great a speed at entry. For getting at a suitable shape, and the best position two or three trials might be necessary.

The shape of the rotary may be circular, elliptical, turbine (Fig. 2), or tangent (Fig. 3).

8.4. Circular Shape

This type suits where roads of equal importance intersect at nearly equal angles and carry nearly equal volumes of traffic. It has the shortest perimeter for a given area. Therefore, the extra distance, over which vehicles are forced to travel across the intersection, is a minimum.

8.5. Elliptical Shape

Sometimes site conditions compel the use of an elongated shape. In such cases, the elongation should be in the direction of the greater flow so that the through traffic moving in the direction of elongation travels practically the same distance as on the straight. The greater the elongation the greater the check the island imposes on cross traffic and the easier the travel on the through route. Therefore, such a shape can be used only at intersections where the cross traffic is small. If the island is elongated too much, the main traffic will be enabled to travel very fast and, therefore, there will be a large difference between the speeds of the traffic of the major and minor roads with the attendant hazards. Therefore, it is necessary that the elongation is so limited that the radius of curvature at the sharpest point is at least 50 feet.

8.6. Turbine Shape

The main feature of this design is that the traffic is forced to slow down at entry by a sharp left-hand turn and is encouraged to accelerate when leaving by providing a tangential exit, Fig. 2.

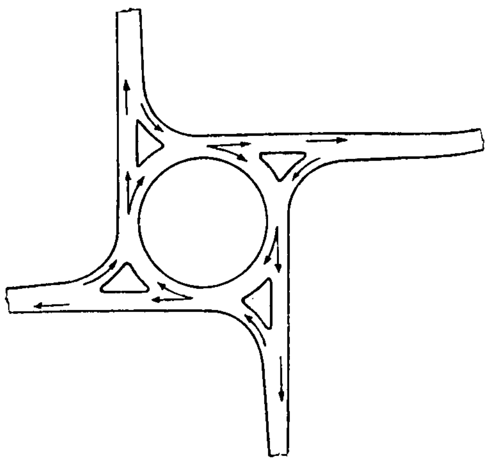


Fig. 2. Turbine type rotary

The advantages of this type are: (i) forced reduction in the speed at entry and full scope for acceleration while leaving; (ii) ease of operation for the vehicles, there being no reverse curves.

The disadvantages on the other hand are: (i) the full left turns impose severe restrictions on the traffic; (ii) the entry to the rotary at night has to be made in the glare of the headlights of vehicles leaving the rotary on the tangential exit; (iii) considerable lengths of all the intersecting roads may have to be re-aligned; (iv) the type does not suit when the number of intersecting roads is more than four.

8.7. Tangent Shape

In this type Fig. 3, one or more of the intersecting roads enter and leave the rotary in a direction tangential to the central

island. Such a layout is not desirable as the traffic on the tangential road would travel at an unrestricted speed much higher than that on the other roads with the consequent hazard.

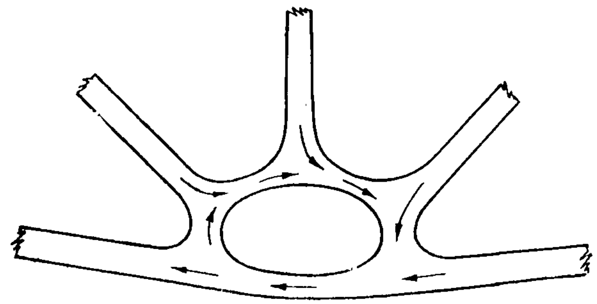


Fig. 3. Tangent type rotary

8.8. Site conditions may require adjustment in the shape of the island and it may also be necessary to re-align one or more of the intersecting roads to ensure the minimum weaving lengths and suitable weaving angles.

8.9. Economic use of land, consistant with the requirement of securing smooth flow of traffic, should be aimed at in evolving the shape and size of the island.

8.10. By shifting the position of the central island and re-aligning the intersecting roads where necessary, it is possible to evolve the best shape for the central island. The trials are best done on a large-scale site plan showing the existing roads, buildings, and other fixed features, as well as controlling levels.

9. RADIUS OF CURVATURE OF THE ROTARY ONE-WAY ROAD

9.1. When a vehicle moves on a circular curve at uniform speed

$$e+f=\frac{V^2}{15R}$$

where e = superelevation of the road surface,

f = coefficient of friction,

V = speed of the vehicle in m. p. h., and

R = radius of the curve in feet.

Adequate superelevation can rarely be given on rotary roads. It would, therefore, be safer to neglect superelevation altogether. So the radius of the central island is given by the equation—

$$f=\frac{V^2}{15R}$$

i.e., $$R=\frac{V^2}{15f}$$

9.2. As a result of many tests by Prof. Moyer it has been deduced that the coefficient of friction on clean wet pavements, at impending side skid, varies from about 0.8 at 10 m. p. h. to about 0.5 at 40 m. p. h. To provide for worse road conditions than those that existed during Moyer's experiments, and to allow for the fact that a vehicle has to be on a rotary with many other vehicles for a longer period than at curves in the open, a factor of safety of 1.5 is considered justified. On these assumptions, the values of 'f' for design speeds of 20 and 25 m. p. h. are worked out below:

Rotary design speed m. p. h.	20	25
Minimum coefficient of friction*	0.7	0.65
Value with factor of safety 1.5	0.47	0.43

9.3. The safe minimum radii for the rotary road are derived as under:

TABLE 1

Minimum radii of rotary roadway

Rotary design m. p. h.	20	25
Coefficient of friction (f) (See para. 9.2)	0.47	0.43
Radius $R=\frac{V^2}{15f}$	57	97
Recommended Minimum Radii in ft	**60**	**100**

9.4. The centrifugal ratios, for the above speeds and radii, work out to 0.44 and 0.42. These values are in excess of 0.25 which is considered comfortable for travel on curves in the open. It is held that in turning movements on rotaries, which are anticipated, the passengers subconsciously prepare themselves and so the greater values are not objectionable.

* Obtained by interpolation between the values given by Prof. Moyer.

9.5. Even though the radii derived in para. 9.3 are for the rotary roadway, in practice it is convenient to design the central island to conform to the above radii.

10. WEAVING ANGLE AND WEAVING DISTANCE

10.1. By tracing the motion of a vehicle entering the rotary from one intersecting road and leaving by another intersecting road, it will be noticed that the vehicle first merges into the one-way traffic flow round the central island and then weaves across to emerge from this flow to the required road outlet. During this operation the vehicle effects a change in its direction of motion in a certain length of travel.

10.2. For a given design speed, the freedom of movement on a rotary depends on the size of the weaving area. If the length of this area is small or its width or radius of curvature restricted, traffic may slow down to the extent that spacing of vehicles will become too little for any weaving or crossing, and the rotary will become congested, causing traffic jams on the entry roads.

10.3. The weaving angle is the angle between the path of a vehicle entering the rotary and that of another vehicle leaving the rotary, and therefore crossing the path of the former. In Fig. 4, GH gives the direction of motion of a vehicle as it merges into the flow and EF the direction as the vehicle emerges out of

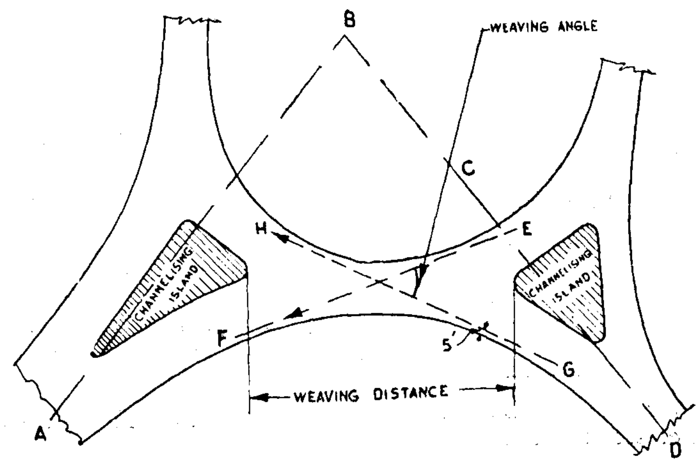


Fig. 4. Weaving angle & weaving distance

the flow on to the adjacent road. The angle between these two lines is the weaving angle. Theoretically, the lines GH and EF are the common tangents of the curves parallel to the central island and the curves at entry and exit at a distance of 5 ft from the curbs as shown in the Figure. This 5 ft is the assumed distance of the centre line of motion from the curbs. In actual design the tangents are drawn at the curb lines as the procedure is easier and no great error is involved. The weaving angle is different from the angle of intersection of the two radial roads, i. e., the angle between AB and CD in Fig. 4.

Observations have shown that some vehicles may have to stop or nearly stop for others to pass in front of them when the weaving angle is large. **For the smooth flow of traffic, the weaving angle should be small.** However, the weaving angle need not be less than 15 degrees because designs attempting small angles involve the adoption of very large diameters for the central islands and are, therefore, costly to construct.

10.4. Weaving Distance

The operation of turning from one road to the adjacent one is effected in the length between the junctions of the two roads with the road round the rotary. In practice, this length, called the weaving distance, is measured along the centre line of the one-way road between the noses of the two channelising islands (Fig. 4), although in actual use the length in which vehicles may be able to weave in and out may be shorter.

Vehicles using a weaving section are of two classes, (1) those entering, passing through, and leaving the weaving section without crossing the path of other vehicles; and (2) those that must cross the paths of other vehicles. It is the latter class that requires greater consideration. A detailed study of weaving practice has been made at the Yale University, U.S.A. For the purpose of this study, weaves have been divided into two main types :

(a) **Optional type of weave,** wherein the weaving vehicle voluntarily moves from a trailing position behind another moving vehicle into a parallel traffic lane.

(b) **Forced type of weave,** in which the weaving car is forced into another traffic lane by the presence of either a stationary object in the roadway, e.g., a parked car, or a slow-moving bullock cart.

Both of the above types are further subdivided into four main classes of weaves :

Free weave, in which the weaving vehicle is influenced only by the vehicle in the lane directly ahead of it.

Retard weave, where the weaving vehicle must wait for another car to move ahead in the parallel lane before the weave manoeuvre can be commenced. In this instance, the weaving vehicle must frequently slow down to the speed of the car directly ahead until the retarding vehicle has cleared sufficiently to permit weaving.

Conflict weave, in which the weaving vehicle moves into the parallel traffic lane directly in front of another car moving in that lane.

Gap weave, where the weaving vehicle moves into the restricted space between retarding and conflicting cars. This type is the

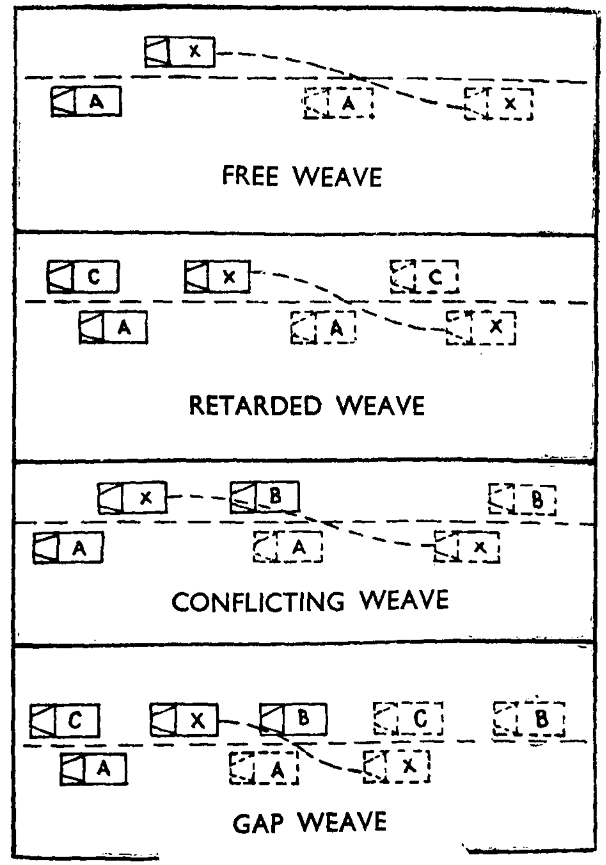


Fig. 5. Types of weaves

Broken Lines Show Start-of-Weave Positions ; Solid lines show End-of-Weave Positions most important in determining the opportunities for weaving under various conditions of speed and traffic volume.

Fig. 5 shows these types diagrammatically where X is the weaving vehicle; A is the vehicle directly ahead of X either moving or stationary; B is the conflicting vehicle; C is the retarding vehicle.

10.5. The Table below gives the average values observed in the Yale University study.[3]

TABLE 2

Average observed values of weaving at the Yale University

	Optional Weaves				Forced Weaves			
	Free	Retard	Conflict	Gap	Free	Retard	Conflict	Gap
Length of weave ft	221.9	167.4	260.7	217.7	131.9	123.5	168.6	145.2
Average speeds at start of the weave m.p.h	35.8	33.3	40.6	37.1	34.4	33.4	39.3	34.9
do do at end of weave m p.h	36.0	33.3	41.0	37.1	32.7	31.2	43.7	32.1
Weave time in seconds	4.1	3.3	4.3	3.9	2.7	2.6	2.9	3.0
Lateral movement ft	9.6	8.1	9.6	9.0	—	—	—	—
Overtaking time seconds	11.3	5.8	14.7	9.0	—	—	—	—

10.6. Experience and observation have emphasised that the effectiveness of the weaving area is more dependent upon the available length than upon the available width provided the latter is above a certain minimum.

10.7. Calculating Weaving Length

Even though observations of weaving have been made in western countries, so far no accepted method has been evolved by which one can calculate the minimum weaving length for a given set of conditions. The following two methods are recommended for rough calculation.

Method I. Any weaving movement can be considered as a change in the lateral placement of a vehicle when it is moving forward at a constant speed. Although the actual path of the vehicle making such a manoeuvre on a straight road forms a transitional curve throughout, for practical purposes it may be considered as a reverse circular curve (Fig. 6).

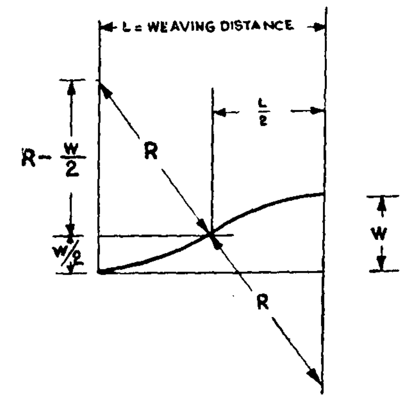


Fig. 6.

From the geometry of the figure it may be shown that:

$$L = \sqrt{(RW - W^2)}$$

where L = weaving length on the straight,

W = transverse displacement of the vehicle,

R = radius of curvature of the path of the vehicle.

In the weaving motion of a vehicle on the inner track leaving a rotary with four radial roads intersecting at right angles, the transverse displacement W, is roughly equal to the radius, R, of the central island (Fig. 7). Substituting this value in the above formula:

$$L = \sqrt{3} \times R.$$

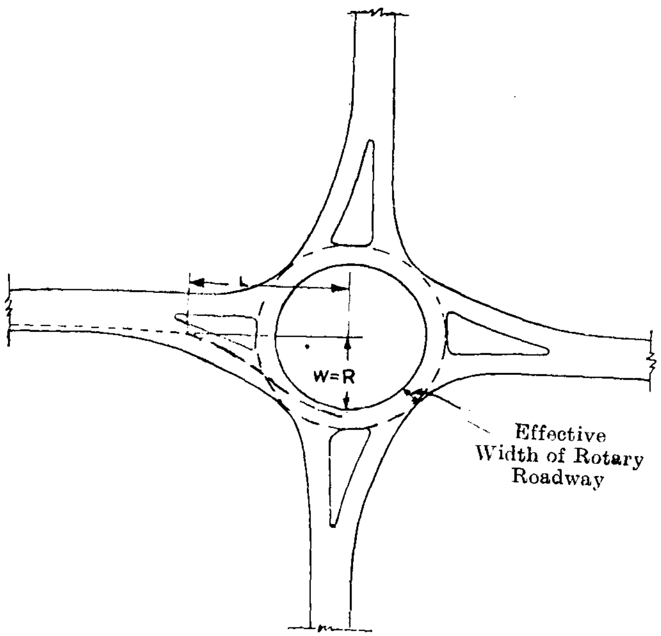


Fig. 7.

Method 2. In the second method, the time taken by a vehicle to move laterally under actual traffic conditions is taken as the basis of calculation. A series of rough observations made in the U.S.A. indicated that a vehicle moving on a straight road would take 3 to 4 seconds to move laterally to an adjacent lane without causing discomfort to passengers and further that this time interval was practically constant for speeds ranging from 30 to 60 m.p.h. It was also observed that about one second more was required for each additional lane width. The Yale University Study, Table 2, also indicated that 4 seconds is reasonable time for optional weaves.

10.8. The American Association of State Highway Officials in their "Policy on Rotary Intersections" give the following table of minimum weaving distances:

Rotary design speed m.p.h.	25	30	35	40
Minimum weaving length ft	150	180	210	240

The above recommendations are in general agreement with the values for the optional weaves observed at the Yale University.

10.9. Under conditions of mixed fast and slow moving vehicles which prevail in India, the effect of a bullock cart may for all practical purposes be considered as forcing a weave. Also, its influence on the weaving length may be neglected because the cart will travel only a very short distance during the weaving time of the motor vehicle.

10.10. In the absence of data derived from observation on Indian roads it is considered that four seconds is a correct time for weaving. The weaving distance, L, will then be the distance travelled by the vehicle in four seconds at the design speed, viz., $L = 88/15 \times V$, where L is in feet and V in m. p. h.

Table 3 shows the weaving lengths, worked out by the two methods described above for the recommended design speeds.

TABLE 3

Calculated and Recommended Minimum Weaving Lengths

Rotary design speeds in m. p. h. $= V$		20	25
Min. radius of central island, R	ft	60	100
Weaving length:			
(1) $L = \sqrt{3}\ R$ ...	ft	104	173
(2) $L = \frac{88}{15}\ V$ ...	ft	116	148
Recommended weaving length minimum ft		**100**	**150**

10.11. **Maximum Weaving Length**

The provision of a minimum weaving length is necessary for preventing congestion on the rotary. If, however, it is too great, vehicles would tend to accelerate when volume of traffic is small and to wander about the rotary carriageway causing traffic hazards.

10.12. Provision of a distance between the adjacent roads slightly greater than the recommended minimum would improve the efficiency of the rotary. But excessive weaving lengths would increase the overall dimensions of the rotary and hence the cost of construction and maintenance. Traffic rotaries should not, therefore, be unnecessarily large. The maximum weaving lengths may be taken as twice the minimum. The following Table sums up the recommendations :

TABLE 4

Weaving Lengths

Rotary design speed m. p. h.	20	25
Minimum weaving length ft	**100**	**150**
Maximum weaving length ft	**200**	**300**

10.13. In any particular case the actual lengths to be adopted will depend on the angles between the intersecting roads and their relative positions. In unsymmetrical intersections, the minimum weaving length may be possible of adoption for some of the intersecting roads and somewhat greater lengths may be required for others. Sometimes it is possible to increase the weaving lengths by slightly realigning the intersecting roads if site conditions permit.

11. WIDTH OF THE ROTARY ROADWAY

11.1. The rotary roadway is a one-way road round the central island carrying for a short distance all the traffic from the intersecting roads. Because the outer curb-lines follow the inlet and exit sides of roads, the actual width of the rotary roadway varies from section to section. An imaginary line can be drawn parallel to the curb forming perimeter of the central island, as shown in Fig. 7. The narrowest width of the area enclosed by this line and the curb of the central island may be said to be the effective width of the rotary roadway. The expression "width of the rotary road" is used in this Paper to denote this effective width.

11.2. As the width of the rotary roadway largely determines the capacity of the rotary, it should be a function of the joint capacities of the intersecting roads. It may be considered in terms of the number of traffic lanes required to accommodate the combined traffic. The traffic using four lanes of, say, two intersecting roads can be accommodated in one lane of the rotary as shown in Fig. 8. Thus the width of the rotary roadway should be about one fourth the total width of all radial roads meeting at the rotary.

11.3. An empirical method of fixing the minimum width of the rotary roadway is to provide for at least two traffic lanes, one for vehicles passing through and one for vehicles making a turn.

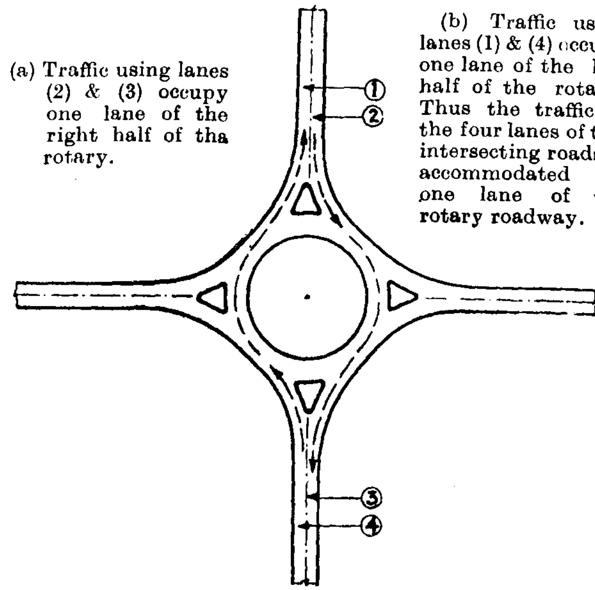


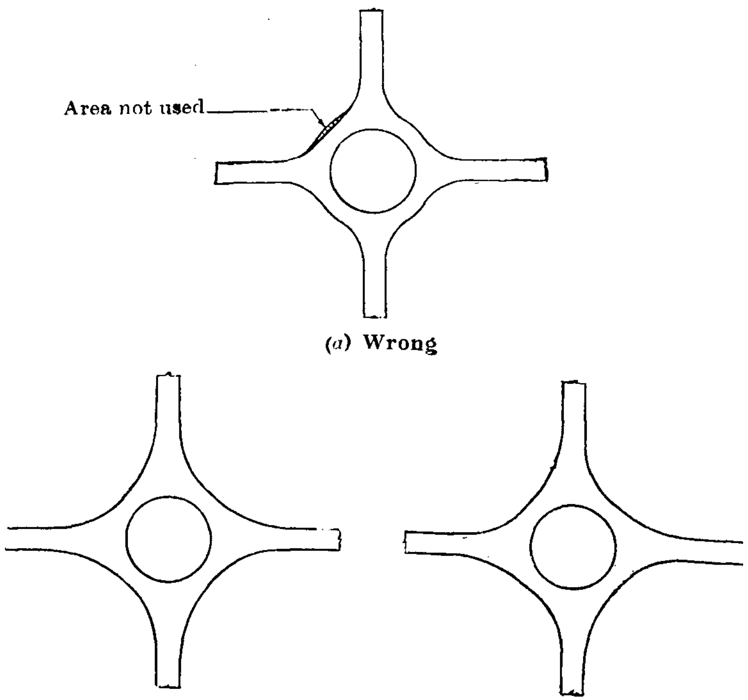
Fig. 8.

In any case, the width is generally made not less than half the width of the widest radial road plus width of one lane. While the A.A.S.H.O. Policy contemplates 24 ft as the minimum width of a rotary one-way road, the authorities in the U.K. lay down 30 ft as the minimum width. Considering that a rotary is generally built where at least one of the intersecting roads is important and carries heavy traffic, **30 ft minimum width is recommended.** A width much in excess of traffic needs results in the wandering of vehicles when the traffic is light and this is hazardous. Economic considerations also point to the desirability of using only the minimum space and thus limiting the width of the rotary roadway to the minimum needed by traffic.

11.4. The minimum of 30 ft recommended will provide three lanes, out of which one lane should be generally available for overtaking. Such a provision would, to some extent, mitigate inconvenience and risk when bullock carts and cyclists mix up with motor traffic on the rotary.

12. OUTER CURB LINE

12.1. The outer curb line of the rotary provides a link between the adjacent intersecting roads. Therefore, it should be designed with the main object of avoiding reverse curvature in the path of a vehicle moving from one radial road to the next adjacent one. Vehicles do not use the shaded portion indicated in Fig. 9(a). The alignment of outer curb as indicated in Fig. 9(b) or 9(c) is correct and should be adopted as far as site conditions permit.



(a) Wrong

(b) Correct

(c) Correct

Fig. 9.

12.2. Where the angle between the adjacent intersecting roads is large and the site conditions do not permit of re-alignment of these, an outer curb line concentric with that of the central island for a short length is unavoidable.

13. CURVES AT ENTRANCES AND EXITS

13.1. The minimum turning radius of a vehicle is the radius of the sharpest curve that could be traced by its outer front wheel. It varies with the make and wheel base of the vehicle and is generally given by the makers for a speed of about 5 miles per hour. But the actual turning movements on roads are performed at about 20 m.p.h. When a vehicle is turning, the rear wheels do not follow the tracks of the front wheels and the track followed by the inner rear wheel normally determines the radius and shape of curve to which the kerb line is set. Observation has shown that kerbs set to a circular curve of 55 ft radius will enable buses and trucks to negotiate a right angle turn easily.

13.2. A vehicle entering the rotary has to decelerate to the design speed of the rotary. Therefore, the radius of the curve at entry should be the same as the recommended minimum radius for the central island for the chosen design speed. Vehicles leaving the rotary would accelerate to the speed of the exit road and, therefore, the exit curve should be of a large radius.

13.3. On theoretical considerations, entrance and exit curves should be transitional but in practice they may be circular or three centred according to the importance of the road and the limitations of site conditions.

13.4. The Ministry of Transport in the U.K. recommend (Memorandum No. 575) that the radius at entry should be 60 ft and that at exit should be 150 ft. In the report of the Departmental Committee on the Design and Layout of Roads in Built-up Areas, this recommendation has been accepted for the outer parts of built-up areas, but for fully built-up streets 35 ft and 70 ft are recommended as the maximum radii of kerbs at entry and exit. Fig. 10 shows the curves of the various radii recommended above.

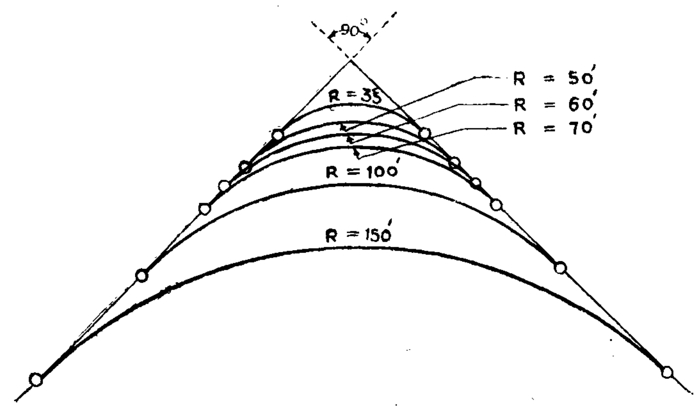


Fig. 10. Curves of different radii for a right angle turn

13.5. **It is recommended that the radius of the entry curve should be equal to the minimum radius of the central island recommended for the design speed.** Wherever practicable, a three centred curve should be adopted.

13.6. Radius of Exit Curve

When a vehicle is leaving the rotary, its speed will be somewhere between the speed on the rotary and the speed on the straight. The radii of the exit curves should, therefore, be somewhat greater than those at the entry; $1\frac{1}{2}$ times is a reasonable figure. It is therefore recommended that **the mimimum radii of curves at exit may be 100 ft and 150 ft for design speeds of 20 and 25 m.p.h. respectively.**

14. PAVEMENT WIDTHS AT ENTRANCES AND EXITS

14.1. The width of carriageway at the entrance and the exit of the rotary is governed by the amount of traffic entering and leaving the rotary. These widths should be based on the ultimate width of the radial road. The amount of widening required at curves is given by the formula[17]

$$We = N\frac{200}{R} + \frac{V}{\sqrt{R}} \quad \text{where } We = \text{extra width in ft}$$

R = designed radius in ft

V = speed in m.p.h.

and N = number of traffic lanes.

The Table below gives the minimum extra width for the minimum entrance and exit radii for the design speeds of 20 and 25 m.p.h.

TABLE 5

Minimum Widening at Entrances & Exits

Design speed of the rotary m.p.h.	20		25	
Radius R ... ft R	60	100	100	150
Extra widening per one lane $We = \frac{200}{R} + \frac{V}{\sqrt{R}}$ (Ft rounded)	6.0	4	4.5	4

Thus when the radius is 60 ft at the entry and the radial road is a four-lane highway, the width at the entrance, i.e., the width between the channelising island and the curb, should be $\frac{44}{2} + 2 \times 6 = 34$ ft.

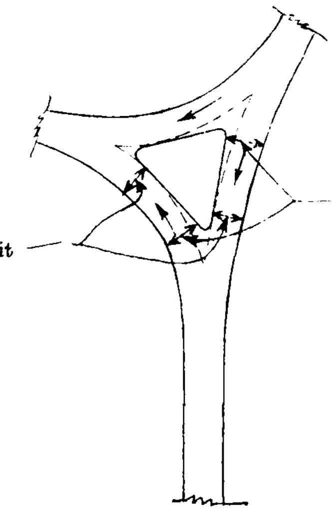


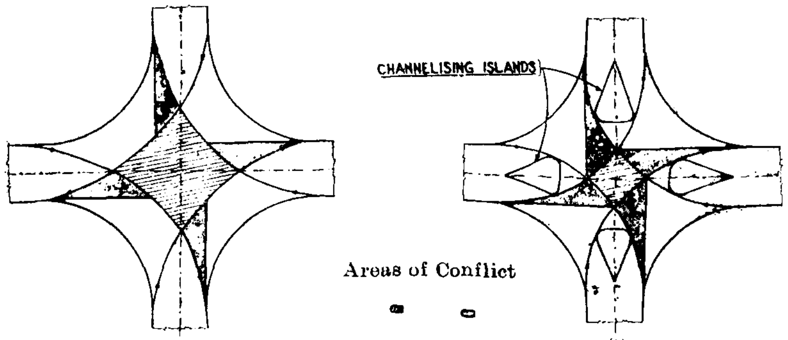
Fig. 11. Channelising island with wide entrance and narrow exit

14.2. Under Indian conditions of mixed traffic, it has been observed that slow moving vehicles cluster at the approaches to a busy intersection and hinder traffic; so it is always desirable to aim at providing a minimum width of carriageway equivalent to two lanes at the entry and exit of a rotary. This can be achieved by widening as shown in Fig. 11.

15. CHANNELISING ISLANDS

15.1. Importance and Necessity of Channelization

Through channelization the area of conflict between intersecting traffic streams can be reduced. Figures 12 (a) and (b) illustrate this clearly. Also, the angle at which the conflicts occur can be controlled. So, channelization is the method suited for promoting safe and orderly movement of intersecting traffic where more expensive methods are not justified.



Areas of Conflict without Channelization. Fig. 12 Areas of Conflict with Channelization.

The functions of channelising islands are:

(1) prevention and reduction of undesirable weaving and manoeuvring by keeping the vehicles to clearly defined paths;

(2) provision of convenient places for erecting signs and lighting the roadway;

(3) helping to establish the desired angles for the crossing of vehicle paths;

(4) permitting streams of traffic moving in the same general direction to converge at very small angles thus minimising possible conflicts;

(5) helping to discourage the prohibited turns; and

(6) serving as pedestrian refuges.

At a rotary the channelising island serves to reduce the speed of the vehicles and helps to indicate to the drivers, not accustomed to the area, the presence of the island.

Therefore, considering the above mentioned advantages, it is very necessary that channelising islands should be provided at entries and exits of a rotary.

It might be urged that these islands occupy space and tend to reduce the free flow of vehicles. We can do no better than quote American experience[6] in this regard.

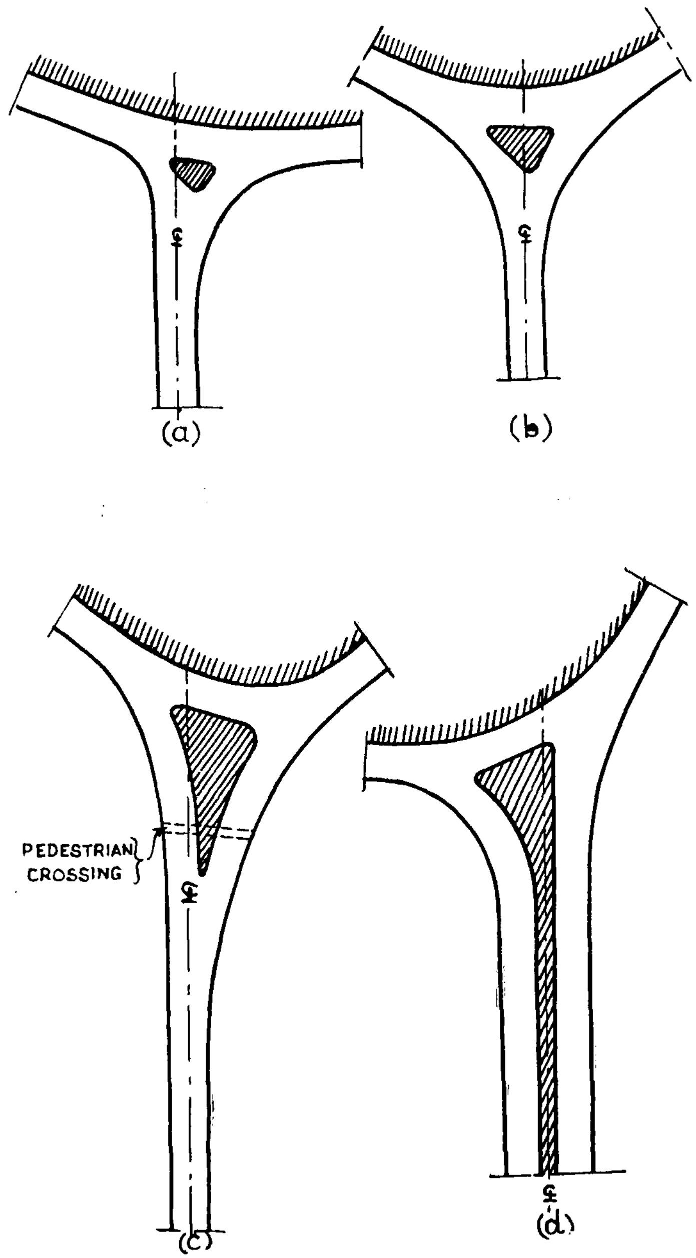


Fig. 13. **Channelising island**

"Not so long ago, obstacles of any form in the roadway were considered pure hazards. However, over the past 10 years or so, during which extensive field tests were made, the traffic engineer gradually reversed his thinking.........

"Of the three basic agents creating intersectional conflicts, the vehicle, the roadway, and the driver, the latter is probably the most important and most changeable and, by extension, so is the vehicle which he controls.........

"In his efforts to control the driver and indirectly manage the vehicle, the highway engineer incorporates in his design certain disciplinary features which not only influence the shape of the roadway, but which also serve to provide more capacity by restricting freedom of movement. At first glance, this seems like a contradiction in terms. Upon closer examination, however, the principle that too much freedom leads to erratic movements and dangerous friction is readily demonstrable as sound."

In many cases, the use of channelising islands as refuges for pedestrians while crossing busy intersections has tangibly demonstrated the need for such safety amenities.

15.2. Shape

The shape and size of a channelising island located at the junction of a radial road with a rotary is governed by the radius of the rotary, the radii of the entrance and exit curves and the angle of the radial road with the rotary. The modern trend is to have either straight sided islands or curves not concentric with the outer curves so as to form funnel shaped entrances towards the direction of motion (Fig. 11). It is, therefore, seen that channelising islands are not symmetrical about the centre line of the radial road. Different designs of channelising islands are indicated in Fig. 13.

15.3. Curbs

Islands are generally provided with curbs to prevent motor vehicles running on to them. Where the channelising island is also to be used as a pedestrian refuge, an effective obstruction to motor vehicles should be formed by the construction of curbs 7 to 9 in. high. A sloping curb, 4 to 6 in. high, may permit a motor vehicle occasionally to mount the island in an emergency. In India, where slow moving animal-drawn vehicles form a very large part of the total traffic, it has been observed that bullock carts easily mount the sloping curbs. It is, therefore, recom-

mended that curbs 6 in. high, as shown in Fig. 14 be used for the channelising islands and the central island.

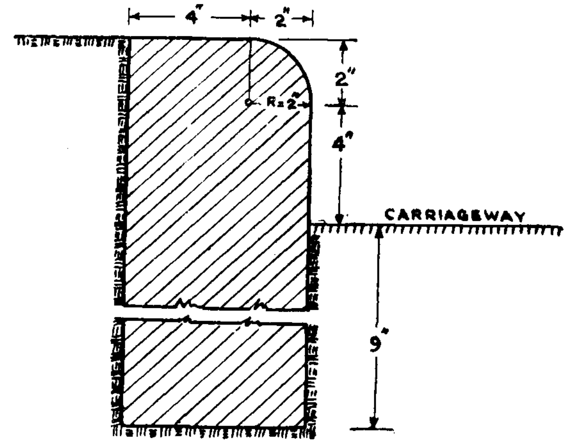


Fig. 14. Concrete curb

15.4 After fixing the width required at the entrance or exit, with due consideration to the number of lanes and the extra width to be provided, a preliminary outline is drawn as indicated by dotted lines in Fig. 11. The straight lines are then fitted in and the corners rounded. Generally, where the traffic is heavy it is desirable that the tail of the channelising island is continued on the radial road for some distance, as shown in Fig. 13 (d), so that it serves as a separating central strip. This would also prepare the motorist for the turn he has to take and for the reduction in speed at the turning. Where such an elongated channelising island is provided it is desirable to flare the approaches to the rotary.

15.5. When traffic on a radial road is light, the channelising island at its entrance to and exit from the rotary may be omitted or constructed without kerbs, the area being delineated by colour contrast by planting grass.

16. CAMBER AND SUPERELEVATION

16.1. A vehicle passing through a rotary traverses a reverse curve. When making a sharp reverse curve the vehicle is subjected to centrifugal forces acting successively in opposite directions and consequently experiences " throw " which may be hazardous in the case of top heavy trucks or buses. The intensity of the "throw" is minimised if the centrifugal force is counteracted by cross slope. The cross slope is minimum at the point of change of direction.

16.2. It may be assumed that the inner portion of the rotary roadway is used by vehicles going through to the opposite side or turning right. The cross slope designed to counteract the centrifugal force of vehicles using the inner portion of the rotary roadway is opposite to the cross slope required for vehicles at entry and exit. The meeting of these cross slopes produces 'a crown line', shown in Fig. 15. The crown line should as far as possible be located so that vehicles will cross it while travelling along the common tangent to the reverse curve. If this is done the centrifugal force and the effect of cross slope will not be in the same direction at any point. Otherwise, the two together may act cumulatively accentuating the "throw" and possibly giving the driver a feeling of unbalance in the vehicle which may result in erratic and unsafe driving.

16.3. As most vehicles, except those turning to the left, have to cross the crown line, cross slopes adjacent to the crown line should not exceed 1/25 to 1/20 and the algebraic difference of the cross slopes on either side of the crown line should lie within the range 1/20 to 1/17 according as the weaving distances are long or short.

16.4. Channelising islands should be situated on the peak with the road surfaces sloping away from them to all sides. This accentuates the channelising function. Whenever possible, the cross slope at an entrance should be carried around on the outer edge of the rotary to the adjacent exit, altering the slope slightly to suit the curvature in the rotary and the exit. This is specially desirable where a good proportion of the vehicles entering by a radial road have their exit to the adjacent radial road, as such vehicles do not ordinarily have to make reverse curves.

16.5. In all cases of changes in the cross slopes, the transition should be gradual. Fig. 15 shows a typical disposition of cross slopes in a rotary.

17. SIGHT DISTANCES

17.1. Rotaries in open or semi-urban areas should be so located that the presence of the channelising islands is easily seen by the approaching vehicles. The sight distance should be as

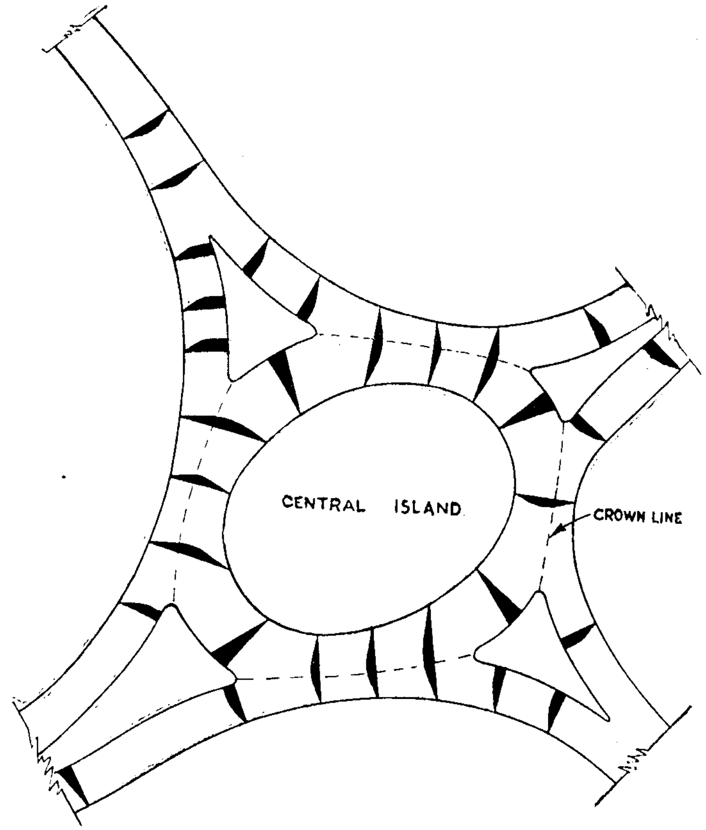


Fig. 15. Cross slopes at a rotary

large as possible but should not be less than the safe stopping distance recommended by the Specifications and Standards Committee[16] for the design speed of the rotary and reproduced below.

Design speed of rotary (m. p. h.)	20	25
Minimum sight distance ft	120	160

In towns it may not always be feasible to secure the sight distances. In such cases, ample warning should be given of the existence of the rotary (see para. 21.1.).

18. GRADES

18.1. It is preferable though not essential that a rotary should be located on level ground. It may be sited to lie on a plane which is inclined to the horizontal at not more than 1 in 50. It is, however, not desirable that a rotary lies in two planes having different inclinations to the horizontal.

18.2. A rotary may, with advantage, be located on a summit. Such locations assist deceleration while approaching and acceleration while leaving the rotary. But it is essential that sufficient sight distance is available.

18.3. Rotaries in valleys always provide a full view to the approaching vehicles, but are likely to induce greater approaching speeds and have drainage difficulties.

19. PROVISION FOR PEDESTRIANS AND CYCLES

19.1. Pedestrians are extremely vulnerable to traffic accidents. Proper provision of footways and pedestrian crossings at rotaries is, therefore, essential. Generally, pedestrians have a tendency to cut across roads by the shortest paths and so where the road traffic is heavy, the tendency of moving indiscriminately on to the carriageway should be prevented by means of guard-rails. Pedestrian crossings, which are meant to induce the movement of pedestrians in a regulated manner at definite points, should be marked to meet the channelising islands so that these islands can be used as refuges as well, Fig. 13 (c). A minimum width of 6 ft for the footpath is recommended.

19.2. Where the number of cycles does not exceed 50 per hour, they may be allowed to use the rotary roadway. When this number is exceeded, it is desirable to segregate the cyclists by providing a separate track. The cycle tracks should have a minimum width of 6 ft, but preferably 9 ft. When the traffic demands, increments should be in 3 ft widths.

19.3. A typical layout is shown in Fig. 16. Where the channelising island is short, as indicated in Figs. 13(a) and 13(b) the cycle track should be led beyond its tail, as at 'A' in Fig.16. But where the island is long, as in Figs. 13(c) and 13(d), a gap should be left in the island as shown at 'B' in Fig. 16. It is essential that the surface of the cycle track should be at least as smooth as that of the main carriageway.

Where there are avenue trees, it is desirable that the cycle track and the footpath are so located that the tree line separates them.

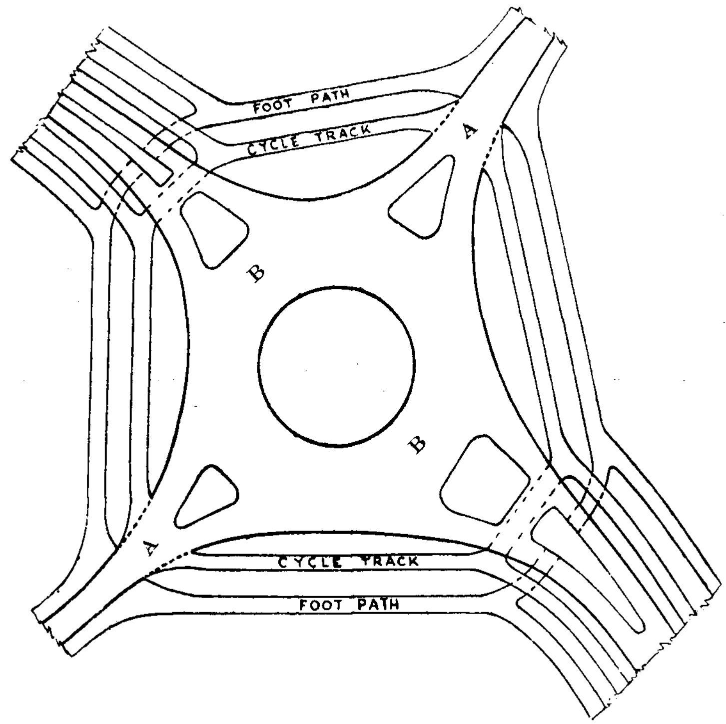


Fig. 16. Layout of cycle tracks and foot paths at a rotary

20. LANDSCAPE TREATMENT

20.1. A rotary provides ample scope for effective development of the landscape. But all such development should only be ancillary to the essential object of traffic control, viz., the reduction in the speed of vehicles and the advance indication of the paths to be followed by vehicles. Planting on the central island should block off the view of approaching headlights so that an impression is not created that a road runs straight through. But once the motorist has entered the rotary, it is desirable that he sees an adequate distance along the chord of the curve to be able to pick off the particular exit road that he wishes to take.

21. SIGNS AND ILLUMINATION

21.1. Vehicles approaching rotaries at night should be well informed of the obstruction to a through run. **A red reflector placed about 3 ft above the road level or a vertical cluster of such reflectors placed 1 to 3 ft high should be fixed on the nose of each directional island, and on the curb of the central island facing the approach roads.** Vertically painted black and white strips, 9 to 12 in. wide, on the curb of the central island and the channelising islands assist visibility at night.

21.2. Exit roads should be indicated by illuminated signs and directional arrows placed both on the edge of the central island and the directional islands, or, in the absence of the latter, at the corners of the exit roads and facing the approaching vehicles.

21.3. In addition to these essential signs, informative signs giving the maximum safe speed may also be given at conspicuous points.

The standard sign indicating the presence of the rotary is given in Fig. 17.

21.4. **Lighting.**

Investigations in the U. K. have shown that rotaries have a much larger proportion of accidents in darkness than other types of junctions. This proves that proper lighting of rotaries is very essential. In this connection, the British Standard Code of Practice for "Street Lighting Part I—Traffic Routes" may be followed. An extract from the code relating to lighting of traffic rotaries is given in Appendix 1.

22. CAPACITY OF A TRAFFIC ROTARY

22.1. The expression "capacity of a rotary" is used to mean the maximum number of vehicles that can pass a given point on the rotary in one hour.

22.2. In the I.R.C. Paper No. 113*, the Specifications and Standards Committee had recommended that 200 bullock carts and 250 motor vehicles per hour may be taken as the capacity of a one-lane carriageway if all the movements were in one direction. Subsequent observations in the U.K. in Central London

*Journal of the I.R.C., Vol. XI-1, November, 1946.

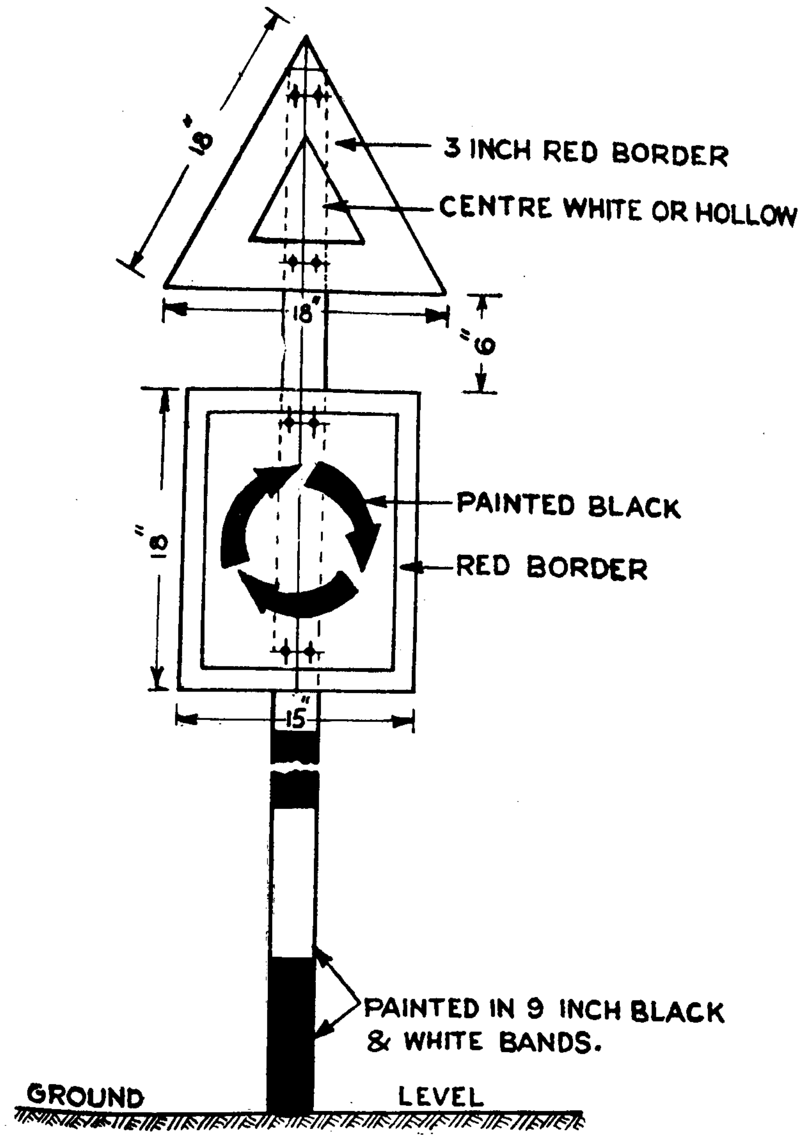


Fig. 17. Cautionary sign—rotary

have shown that the relation between the speed and flow in vehicles per hour may be represented by the empirical formula:

$$Vr = 31 - \frac{q + 430}{3\,(W - 6)}$$

where Vr = mean running speed in m.p.h., or 24 m.p.h. whichever is less,

q = total flow in vehicles per hour,

W = width of carriageway in feet.

This relation is graphically shown in Fig. 18.

22.3. Observations in America on multi-lane highways indicate that for the purpose of computing traffic flow medium capacity trucks (vehicles having dual tyres on rear axles) can be taken as equivalent to two passenger cars on level terrain and to four passenger cars on rolling terrain. Similarly, in mountainous country a truck is roughly equivalent to 8 passenger cars. Since trucks and buses occupy larger space on the road, cover up more vision space, and travel at lower speeds than motor cars, this conversion factor is greater than one depending on the nature of the terrain traversed. Travel at lower speeds than the average by some vehicles increases the number of overtaking manoeuvres. The slow-moving bullock cart also roughly occupies the same area as a truck, but the manoeuvre required for overtaking it is relatively short owing to the greater difference in speed. So it might be assumed that a bullock cart is equivalent to 3 passenger

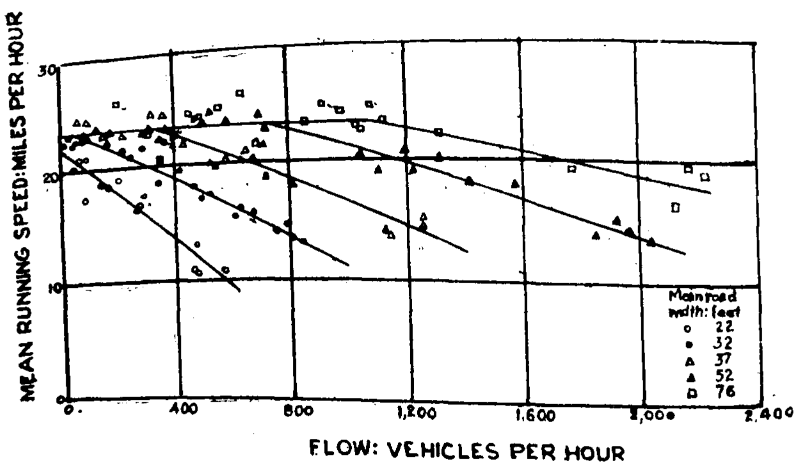


Fig. 18. Relation between mean running speeds and total flow in central London

This relationship is shown in Fig. 19.

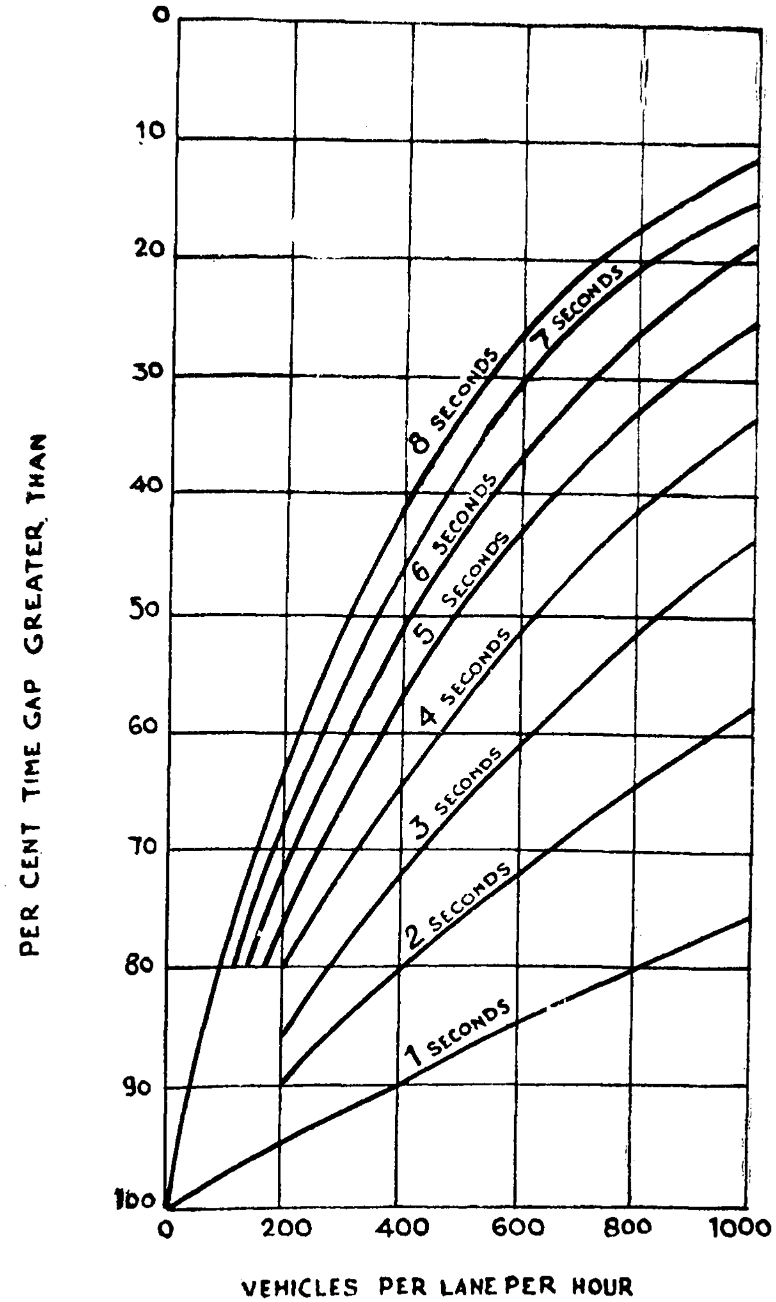


Fig. 19. Frequency of occurence of time gaps of a given duration or greater

cars on level terrain and to 6 passenger vehicles in rolling country. It is recommended that this conversion factor may be applied in estimating the total traffic at an intersection for the purpose of examining the need for a traffic rotary (see para. 3.1.).

22.4. The number of vehicles the rotary can deal with is the sum of the vehicles, which each of the weaving sections can accommodate. Studies in the U.S.A. have revealed that at a channelised intersection where two movements cross each other and there is no appreciable length to weave, 1,200 vehicles per hour is the maximum total of the two movements. Thus for a rotary with four radial roads, the number of vehicles that can be accommodated per hour would be 4,800 vehicles.

22.5. The number of opportunities for merging and weaving of vehicles in a rotary can also be calculated using the Theory of Probability. The Poisson sequation given below, which gives the frequency of occurrence of time gaps of a given duration are longer, can be used for this purpose.

$$P = e^{-\frac{v}{3600}}$$

where P = per cent of time gaps equal to or greater than "t" seconds,

t = acceptable time gap in seconds,

v = number of vehicles per lane per hour,

e = base of Naperian Logarithms = 2.71828.

The percentage openings of the given duration can be obtained by reading against the given number of vehicles per hour corresponding to the selected time-gap. The percentage multiplied by the number of vehicles gives the number of opportunities for merging or weaving.

22.6. The Yale University study [3] (Para. 10.5) has also shown that 1.4 seconds is the minimum clear time spacing between two succeeding vehicles for a gap weave. For a speed of 20 m.p.h. and a car length of 20 ft the time taken to cover the length of a car, will be 0.7 seconds.

From Fig. 19, for a time-gap of (1.4 + 0.7) 2.1 seconds and for a flow of 200 vehicles per hour, 89 per cent of the vehicles would have a gap of 2.1 seconds or more. Therefore, at any one time 0.89 × 200 vehicles alone can weave into and 0.89 × 200 vehicles can weave out of a rotary. This would mean that for

free flow from and to four radial roads, i.e., for a total of eight traffic streams, 0.89 × 200 × 8 = 1,424 vehicles alone out of 1,600 vehicles can weave under ideal conditions. The balance of the vehicles would have to be turning vehicles. Similarly for a flow of 600 vehicles per stream only 70 per cent could weave and the rest should be non-weaving or turning vehicles. The capacity as worked out above applies to ideal conditions when vehicles move systematically with no variations in driver behaviour.

22.7. Actual observations have been made in the U.K. and the Table below gives the results of the observations.

TABLE 4

Observed Capacities of Rotaries

Diameter of island FT	Width of carriage-way round island FT	Total width between outer curbs FT	Maximum converging angle DEG.	Peak capacity in vehicles per hour including cycles
60	30	120	85	2,500
75	30	135	59	3,000
100	30	160	55	3,500
105	40	185	59	4,000
150	30	210	40	4,000
140	50	240	53	5,000
180	40	260	40	5,000
240	50	340	40	6,000

22.8. Observations on the capacity of a rotary having a 100 ft diameter central circular island, two weaving sections of 210 ft each and two weaving sections of 105 ft each, have been made in the U.S.A. [25] The maximum number of vehicles that a weaving section could accommodate varied with the proportions of :

(a) vehicles weaving from the outside to an inside lane,

(b) vehicles weaving from the inside to the outside lane, and

(c) vehicles going through the section without crossing the path of other vehicles.

When all the cars crossed one another, 50 per cent being from the inside lane and 50 per cent from the outside lane, the capacity was observed to be 1,200 vehicles per hour. But when all the vehicles negotiated the section without crossing the path of other vehicles, the capacity was 2,300 vehicles per hour. For ratios of crossing other than 50 : 50 the capacity was between 1,200 and

2,300. It was also observed that larger the number of vehicles passing through the weaving section in one hour the lesser was the speed of operation of the vehicles.

22.9. For Indian conditions, in the absence of factual data, the observed capacties indicated in the Table 4 may be adopted. The capacity indicated may be taken as passenger cars, slow-moving, animal-drawn vehicles and trucks being counted respectively as equal to 3 and 2 passenger cars in level terrain and 6 and 4 in rolling country. A cycle should be counted as equal to one car.

23. UNDESIRABLE FEATURES TO BE AVOIDED

23.1. The provision of a footpath on the periphery of the central island induces pedestrians to cross the carriageway, thereby giving rise to traffic hazards. Therefore, it is not desirable to provide a footpath round the central island.

23.2. A layout of the outer curbs concentric with the central circular island creates a space which is not used by vehicles and is, therefore, wasteful Fig. 9 (a). Also, it necessitates reverse curves for traffic entering or leaving the rotary. So the entry and exit curves should be connected as shown in Fig. 9 (b) or 9 (c).

23.3. Symmetrical entry and exit curves are inappropriate from the point of view of the proper functioning of the rotary. These curves should be of radii as recommended in Paras. 13.5 and 13.6. See Fig. 13(c).

23.4. Channelising islands with straight sides and with entries wider than the exits are more effective than symmetrical islands.

23.5. A staggered intersection can be converted into a rotary with an elongated central island when sufficient area is available as indicated in Fig. 20. Such an arrangement would relieve congestion, specially when the roads are heavily trafficked.

24. TYPICAL EXAMPLES

Plates I to V show typical examples of rotaries recently designed, and some of them constructed, in Delhi. A brief description of the salient features of the design is given in each Plate. Photos 1 and 2 show the rotary figured in Plate IV before and after construction.

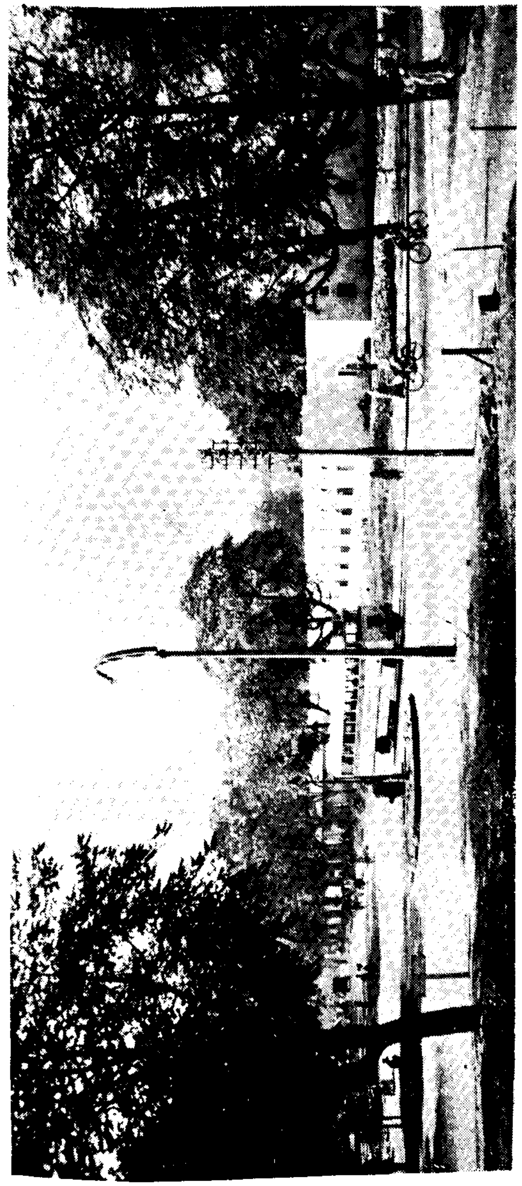


Photo 1. (Before improvements).

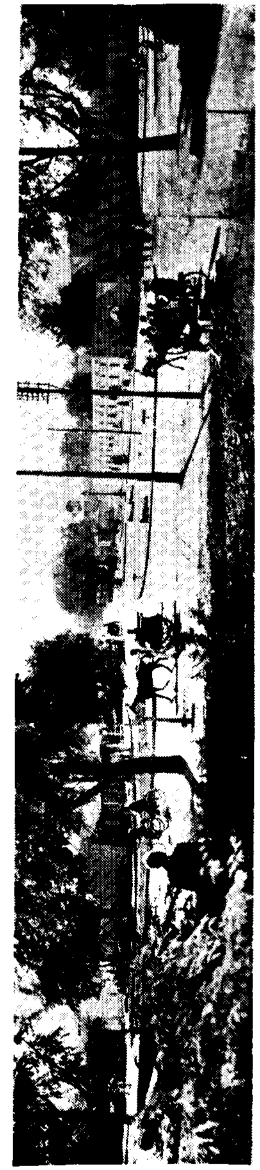


Photo 2. (After improvements).
Junction of Reading Road and Punchkuin Road, New Delhi.

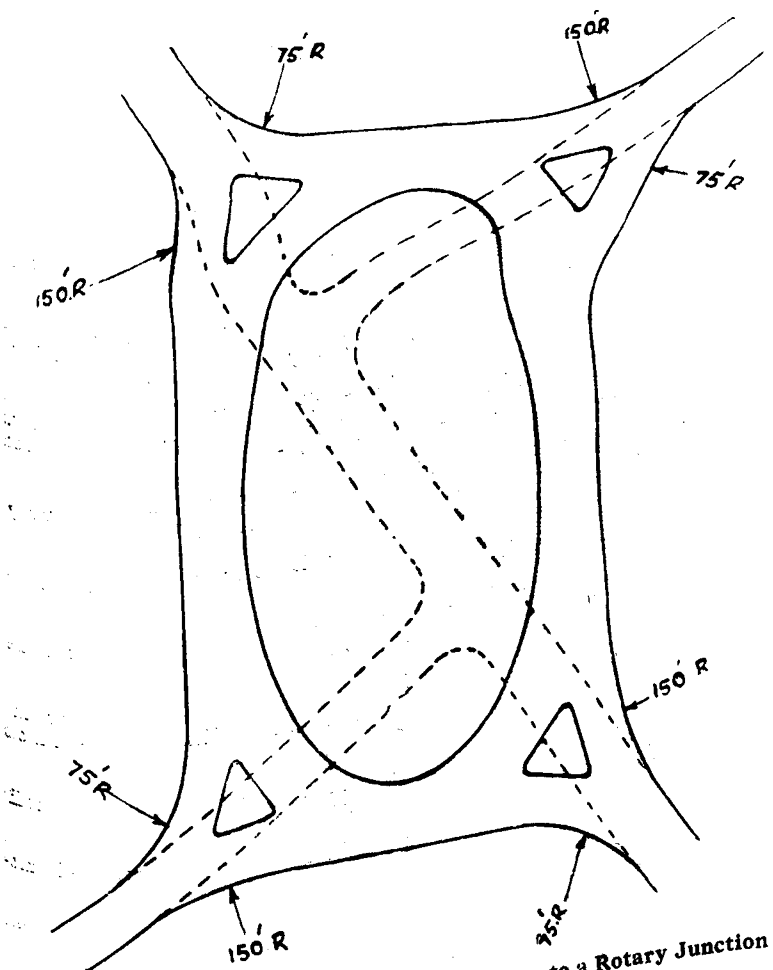


Fig. 20. **Conversion of a Staggered intersection to a Rotary Junction**

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2. A Policy on Intersections at grade - American Association of State Highway Officials - Washington, 1940.
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APPENDIX 1

1. The lighting of roundabouts presents special problems since the movement of traffic is continuous and approach speeds may be high. Two cases may be mentioned.

(i) Where the approach roads are unlighted or are lighted to a poor standard; and (ii) where the approach roads, or the more important of them, are lighted to traffic route standard.

2. Roundabouts in open country are normally of the first type and present the added difficulty that there are usually no buildings or other vertical backgrounds to help the lighting engineer. The use of cut-off sources is essential in these conditions since drivers approaching the roundabout may have been travelling for a long period on unlighted roads. On roundabouts of the second type, non-cut-off lanterns may be used.

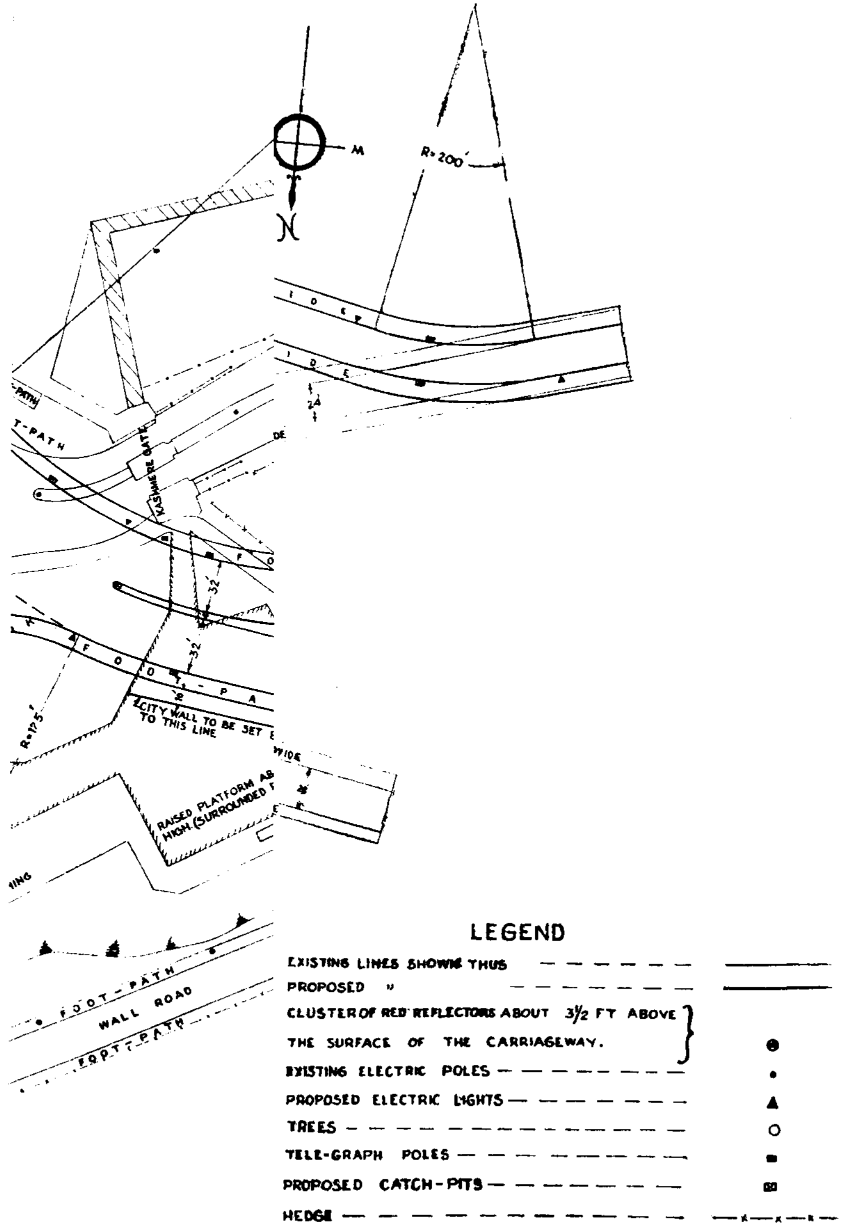
3. Finality has not been reached in the design of lighting instalations for roundabouts, but experience so far leads to certain definite conclusions which are embodied in the following recommendations.

4. If the central island is very small, that is not more than 60 ft in diameter, a satisfactory result is obtained with a single lantern having a symmetrical distribution and mounted centrally at a height of 25 ft or more; a mounting height of 30-35 ft is often advantageous. With this type of lighting it is essential that the lantern should be of the cut-off type, with the angle of cut-off so adjusted that light reaches just beyond the outer edge of the roundabout carriageway.

5. With the single exception described in the preceding paragraph, lanterns should not, in any circumstances, be mounted at the centre of a roundabout.

6. For the more usual roundabout with a central island of up

PLATE H



IMPROVEMENTS TO THE KASHMERE GATE JUNCTION, DELHI

1. HIGHWAYS IN AUSTRALIA
2. THE PLASTIC THEORY AS APPLIED TO THE DESIGN OF MILD STEEL BEAMS AND FRAMES

HIGHWAYS IN AUSTRALIA

By

V. V. GOPALAKRISHNA AYYAR*

CONTENTS

1. ADMINISTRATION

1.1. The constitutional set up in Australia is almost similar to that in India and consists of the autonomous States of Queensland, New South Wales, Victoria, South Australia, Western Australia, the Northern Territory and Tasmania, and the Federal centre at Canberra. Each State is independent within its territorial jurisdiction and maintains its own administrative and executive departments to carry out its programmes. Building

* Shri Ayyar was deputed to study roads in Australia under the Colombo Plan.

projects are generally looked after by the Public Works Department or Housing Commissions, Irrigation and Hydro Electric Projects by the State Rivers & Water Supply Commissions, Electricity Generation & Transmission by the State Electricity Commissions, Water Supply & City Transport by the Metropolitan Water and Transport Boards, and the Highway Systems by the Country Roads Board or the Department of Main Roads. The Chairman and members of the Country Roads Board, Victoria, and the Chief Commissioner and Assistant Commissioner of the Department of Main Roads, New South Wales, are engineers who have risen up from the cadre of Chief Engineers.

2. FUNCTIONS OF THE ROADS BOARD AND FINANCE

2.1. Funds for the Highways are allotted directly by the State and the Federal Governments to the Board which is in sole charge of the administration of the Funds. The main source of money for Highway Development in a State is the customs duty of 10 d. per gallon on imported petrol, and an excise duty of 8½ d. per gallon on locally produced petrol. This amount is collected by the Centre as Federal revenue and is divided among the contributing States on an area-population basis, 2/5th by area and 3/5th by population. Besides this, there is the Vehicle tax, the Motor Registration Fees and the Driver's Licence Fees. All these collections go into the Highway Fund as direct State revenue and are available for expenditure on the improvement of roads under the direction of the Roads Board.

In the State of Victoria, the Motor Tax and Registration Fees are charged on a power-weight basis. A 20 H.P. motor vehicle weighing 20 cwt makes 40 power-weight units and the rate of tax comes to 3 sh. per unit or £ 6 per year. The Registration Fee collected, less cost of collection, goes directly to the Highway authority.

In the State of New South Wales, the system followed is slightly different. Provision is made for the establishment of three types of Funds :

(a) The County of Cumberland Main Roads Fund, the money in which can be spent only on main and secondary roads within the County of Cumberland (Sydney and suburbs), except that a comparatively small proportion can be spent on any public roads or on any works connected with Transport.

(b) The Country Main Roads Fund, the money from which can only be spent on main roads in the area outside the County of Cumberland, except that a comparatively small proportion can be spent on any public roads or on other works connected with Transport.

(c) The Developmental Roads Fund, the money from which can only be spent on the construction of developmental roads and developmental works as distinct from main roads but within regard to the area in which the works are located.

3. ORGANIZATION OF THE HIGHWAY DEPARTMENT

3.1. The Country Roads Board, Victoria, consists of a chairman, 2 members, and one Lay Secretary. The Chief Engineer with his team of officers, comprising one Deputy Chief Engineer, one Plant Equipment Officer, a Bridge Engineer, a Highway Engineer, and six Headquarters Divisional Engineers, a Chief Draughtsman and 10 Territorial Divisional Engineers, works under the Board. The Headquarters Divisional Engineers are (1) the Plants & Survey Engineer, (2) the Asphalt Engineer, (3) the Planning Research Officer, (4) the Mechanical Engineer, (5) the Location Engineer, (6) the Materials Research Engineer. All important designs originate in the Chief Engineer's office. The field work is in charge of the overseer.

3.2. The Highway Engineer is in overall charge of all highway projects and acts as liaison between the Chief Engineer and the various engineers in the Headquarters divisions. The Bridge Engineer controls the design and execution of all major bridge works in the State. Small bridge or culvert works are carried out by the Divisional Engineers themselves under directions from the Bridge Engineer. For exceptionally important bridge works, special Resident Engineers are also employed.

The Lay Secretary looks after all major items of correspondence.

The organization in Victoria is shown in Plate I.

3.3. The State is divided into a number of Municipalities or Shires more or less like Taluks in India. The road system is classified under State Highways, Trunk Roads, Main Roads, and Minor Roads. Each Shire has a Shire Engineer to look after the roads other than the Main Highways falling within its jurisdiction. This is more or less similar to the arrangement in India, where the Central Government is responsible for the National Highways while the States maintain the State Highways and other District Roads. A contribution to the extent of 2/3 of the expenditure is given by the State Highway Department to the

Shire for the construction and maintenance of the Main Highways within the Shires and the work has to be carried out to the satisfaction of the Chief Engineer of the Department. The Divisional Engineers of the Highway Department exercise technical control over the work done by the Shire Engineers.

4. HIGHWAY PLANNING

4.1. Complete traffic studies are made from periodic traffic counts taken in different parts of the country, traffic trends are estimated and future volume of traffic predicted fairly accurately. The design of the Highway pavement is based on such traffic studies and is related to the assumed life of the pavement. The construction work, however, proceeds on the progressive 'stage construction' basis, i.e., in step with the gradual development of traffic. At no time, however, is there need to dismantle any portion of the road structure already constructed in adapting it to the increased traffic. It is only a matter of gradually upgrading the existing road to suit the traffic as it develops.

5. DESIGN STANDARDS

5.1. When fixing the standards for a particular road, the following points are considered:

(a) Traffic density, i.e., the number of vehicles using the road during peak periods. The volume of traffic is given more consideration than the magnitude of the permissible wheel loads. The daily traffic volume is generally assumed as 10 times maximum hourly volume. Rate of traffic increase is ordinarily assumed to be 5 per cent per annum.

(b) Character of the traffic. Passenger vehicles, trucks, or mixed.

(c) Assumed design speed in miles per hour.

5.2. Single lane pavements are adopted where visibility is good, and in flat country where traffic does not exceed 100 vehicles per 12 hours count. Minimum width is 10 ft with shoulders not less than 4 ft wide each. Three-lane roads are not popular since they offer only limited advantage over the 2-lane roads, and are unsafe where the passing sight distance cannot be provided. Four-lane roads are invariably provided with median strips.

5.3. The full land width required for the road, including sufficient width of reserves for ultimate development, is acquired initially because of the lower land values ruling in the initial

stages of any project. Drainage structures are provided for the full width in the first stage and remain undisturbed.

5.4. As regards widening on curves, sight distances and horizontal and vertical curves, the standards laid down by the American Association of State Highway Officials are followed. Horizontal visibility on hill side is provided by benching back the cut so as to provide a minimum sight distance at least equal to the minimum stopping distance measured along the arc of the curve at the centre line of the inner lane.

6. LOCATION SURVEY AND FIELD WORK

6.1. The field reconnaissance work is followed by trial survey and finally by the permanent detailed survey.

Field reconnaissance is done using the prismatic compass, aneroid barometer and the clinometer. Distances are estimated by pacing, or car speedometer or measuring wheel or rotometer.

The Reconnaissance Report generally contains the following information:

(a) Introduction
  (i) Purpose of road
  (ii) Authority to carry out investigations

(b) Summary of Essential information
  (i) As supplied by the Engineer
  (ii) Brought to light by locater

(c) Manner in which field work was accomplished
  (i) Personal
  (ii) Other authorities
  (iii) Local help

(d) Discussions of control points
  (i) From essential information
  (ii) Physical
  (iii) Human

(e) Description of practicable routes.

(f) Conclusions and preliminary estimates of cost of construction of certain routes.

(g) Recommendation of trial surveys.

6.2. Photogrammetry has been very largely employed for obtaining overlapping snaps of the country and contour plans are prepared therefrom using the 'Wild Aerograph' or other similar instrument. The main control points are established by field survey parties.

6.3. In hilly country the trial line is usually run by clinometer and ranging rods and marked by stakes or blazes on trees. The distance is usually chained and typical cross slopes are noted by clinometer readings. The trial line will connect the low saddles with good stream crossings on as nearly as possible a direct line between salient points of topography or population centres to be served; rocky cliffs and country liable to slips are avoided as far as possible. On a long route, if there is for some distance, choice between following the bed of a valley and the top or upper portion of the slopes of a ridge above the river, it is frequently cheaper to locate the road near the ridge, thus reducing the cost of the structures across tributary streams.

The trial line is kept slightly flatter than the final grade to allow for shortening when curves are put in.

6.4. The Permanent Survey

This is carried out on the general location of the most satisfactory trial line, detailed alterations being made to obtain the best crossings of water courses, and to avoid objectionable minor features such as rocky out-crops etc. All curves are set out accurately. In hilly country, it is usual touse the prismatic compass for bearings. Present day practice is to use a light theodolite. The road boundaries are fixed by a theodolite traverse. Centre line is pegged every 100 ft on straights and every 50 ft or less on curves. All tangent points are pegged and reference pegs placed opposite all tangent points and not more than 200 ft to 300 ft on straights.

Following the pegging of the alignment, all pegs are levelled and the cross slopes at each peg taken by clinometer, or, in irregular cross slopes, the cross section is obtained by levels. In hilly country, the levels are taken sufficiently far on each side of the centre line to allow of adjustment of alignment in the office in order to obtain the greatest economy in earth work. The catchment areas of all water courses are recorded where this information has not already been obtained from aerial photographs and the suggested sizes of the culverts are indicated in pencil. At all large creek and river crossings, flood levels and a cross-section of the bed of the stream for a length of 100 ft above and below the crossing are recorded.

The alignment pegged may be adjusted during the preparation of plans. Adjustments may be shown on the plans, or, in some cases, the line may be pegged again.

6.5. Plans and Sections

When complete, these show what has to be imposed on the earth's surface, with reference points which can be identified both on the plan and on the ground. They normally comprise:

Drawing	Scales commonly used	Information shown
Longitudinal Section.	400 ft to 1 in. horizontally and 40 ft to 1 in. vertically.	Levels of ground, finished work at each centre line peg, or at intermediate points, where necessary. Cut or fill at each centre line peg, grades and vertical curves, volumes of cuttings and fillings, location and level of reference pegs and location, length, size and invert levels of culverts.
Cross Sections.	10 ft to 1 in. vertically and horizontally.	Formation level on centre line, C. S. of work required at each centre line peg.
Typical Cross Sections.	Various; say 5 ft to 1 in. vertically and horizontally.	Details of work to be carried out. Pavement and shoulder widths, pavement depth, depths of various courses in pavement. Cross slopes of pavements and shoulders. Cross slope in cuttings and in fills. Side slopes of drains.
Alignment Plan.	400 ft or 200 ft to one in. Larger scales where shifts from pegged line have to be shown.	North point, length and bearing of straights, intersection angles, chainage of tangent points. If transition curves are introduced, value of V = Speed value of curve in m.p.h., L = Length of transition curve, R = Radius of circular arc, and E = Banking on circular arc. All relevant topographical and physical features, drainage lines, structures, fence lines.
Locality Plan.	Various small scales.	Location of work, relative to nearest town or road junction, etc.

6.6. The following are notes on grading the Longitudinal Section.

The levels are plotted to a large scale, i.e., 10 ft to 1 in. vertically and 100 ft to the inch horizontally. As straight lines are seldom found in the natural surface, a series of large radius vertical curves are much more economical and more sightly when well proportioned to the natural undulation of the ground.

Vertical curves are generally computed for the final L.S., but for preliminary grading they are plotted with large French curves with a limiting radius depending on the visibility demanded by the speed value of the section. Cross-sections are plotted from the surveyed cross-sections, and the trial grading generally to a scale of 10 ft to one in.

Only by trial and error can a designing draughtsman be sure that he has secured the greatest economy in grading, and although the process is tedious and laborious, it is well worth while as "it is easier to move earth and rock on a drawing board than with a power shovel."

For the purpose of balancing cuttings and fillings, it is usual to allow a percentage of from 15 to 25 excess of cutting over filling to provide for wastage and shrinkage in ordinary earth using the latter amount for shallow work.

As the exact amount of earth work cannot be computed with certainty to less than 10 per cent in broken country, it is usual to take out the cubical contents of cuttings and banks by mean areas only.

7. CONSTRUCTION

7.1. The road formation work is done by use of earth moving machinery of different types. On a typical road work inspected in Victoria, the following items of machinery were used:

For clearing	—	One Diesel Tractor with bulldozer equipment.
For earthwork	—	Four Tractors, two Power Graders, three Carry-all Scoops of 11 cu. ft. capacity each, and one Air Compressor.

7.2. At the quarry, the site clearance and the bringing up the shale deposit are done by Tractor Bulldozer. The material is lifted and loaded into trucks by a power shovel with a $\frac{3}{4}$ cu. yd. bucket. The shale is conveyed using 12 motor trucks of 6 cu. yd. capacity each with hydraulic tippers. Rolling is done by multi-wheel pneumatic tyred roller driven by a Tractor of 35 H.P. Sheepsfoot roller is used when necessary.

8. SOIL RESEARCH

8.1. A fully equipped Material Research Laboratory is maintained in the head office of the Highways Department under the control of the Materials Research Engineer. Smaller laboratories are maintained, one attached to each Division, to carry out the ordinary tests on local soils and materials. The design of the pavement is done only after correct analysis of the properties of the subgrade soil and base course material.

8.2. C. B. R. from other Test Results

Owing to the need for simplifying tests, the method now commonly used is the identification tests to estimate the C. B. R. The estimate so made can be checked by the actual test in cases of doubt. This method gives the C. B. R. of the material passing a $\frac{3}{4}$-in. sieve in terms of:

A = percentage passing the No. 36 B. S. sieve

B = percentage passing the No. 200 B. S. sieve

C = Linear shrinkage as a percentage

D = Plasticity index

E = Percentage passing the No. 7 B. S. sieve

G = Group index

Three estimates of the C. B. R. are made, using the following formulae:

1. $\text{Log}_{10}\ \text{CBR} = 1.668 - 0.00506A + 0.00186B - 0.0168C - 0.000325BC.$

2. $\text{Log}_{10}\ \text{CBR} = 1.886 - 0.0143D - 0.0045A + 0.0051\frac{B}{A} - 0.0000456\frac{B^2}{A} - 0.00372E.$

3. $\text{C. B. R.} = 4.5 + \frac{(20 - G)^2}{18}.$

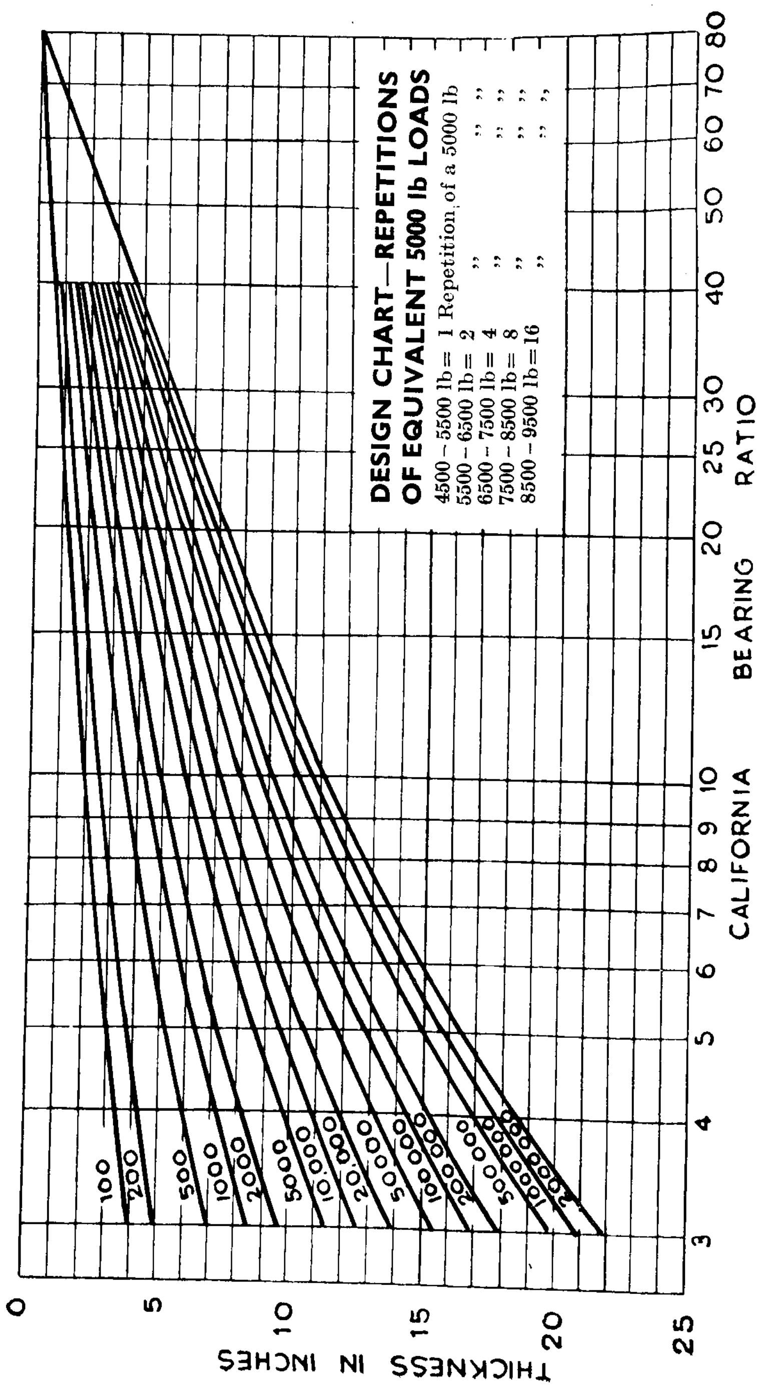


Fig. 1

In practice, tables and charts are used to obtain the estimated value of the C. B. R. If reasonable agreement is not obtained, an actual C. B. R. test is carried out.

9. DESIGN OF PAVEMENTS

9.1. C. B. R. Method

The thickness of the pavement required to withstand the estimated volume of traffic is ascertained from a chart giving the pavement thickness for the various repetitions of the standard 5000 lb load during the life of the pavement, Fig. 1. The number of repetitions is arrived at by taking into consideration the traffic, the week-day factors for the traffic count, the placement factor depending on the width of pavement, the direction of the predominant traffic and the climatic factor depending on the rainfall.

The conversion of the simple traffic count to repetitions of the standard 5000 lb load for a typical rural station is effected by considering the recorded number (A) of light and heavy trucks and also buses (provided the latter are not too numerous) as given below. Cars and utility trucks which generally have wheel loads considerably less than 5000 lb are neglected.

Week-Day Factors

The simple traffic total (A) is usually counted on a day other than Saturday or Sunday for 12 hours (7 a. m. to 7 p. m.). Special counts have shown that, for trucks, the 24 hour total may be generally taken as 1.2 times the 12 hour count. Traffic also varies considerably through the week in accordance with the daily factor F given below.

Day of the week	F	Day of the week	F
Monday	1.03	Friday	1.04
Tuesday	1.04	Saturday	0.51
Wednesday	0.89	Sunday	0.35
Thursday	0.99		

On the basis of an average week-day factor of $F = 1$ for the period Monday-Friday, the average daily traffic for the whole week is $\frac{A}{F}\times 1.20\times\frac{5.0+0.51+0.35}{7}$ approximately $=\frac{A}{F}$. The truck traffic for a year is thus $365\times\frac{A}{F}$.

When the traffic count includes both loaded and unloaded trucks of various types, the number of 'Standard' (5000 lb) repetitions is of the order of 0.7 times this total or approximately $255\times\frac{A}{F}$ a year.

Placement Factor

A study made regarding the habits of drivers in placing their vehicles relative to the edge of the pavement for various pavement widths indicated that the most heavily loaded part is a strip of 3 ft width. The total number of repetitions calculated, as mentioned above, must be multiplied by the following factors to give the number of repetitions on the most heavily loaded strip.

Placement Factors

Pavement width feet	Placement Factor	
	Traffic equally loaded in both directions	Traffic predominently loaded in one direction
16	0.30	0.45
20	0.21	0.35
25	0.16	0.20

The trend of traffic increase was approximately at the rate of 5 per cent per annum. For this rate of increase, the total repetitions are in the following ratios :

No. of years life	1	5	10	15	20
Ratio of total repetitions	1.0	5.5	12.6	21.6	33.1

Climatic Factor

Climatic Factor M = Average annual rainfall of the locality in inches × average number of wet days in the year divided by 10,000. Value may be less than one in dry climates and more than one for very wet.

Where the road is subject to flooding or in some way is liable to be saturated for longer than would be expected merely from climatic factors, an increase in the number of repetitions should be allowed.

Generally, C.B.R. for pavements which are to be sealed 50 min.

L.L.	do.	do. do.	25 ,,
L.L.	do.	not to be sealed	35 ,,
P.I.	do.	are to be sealed	0 to 6 (9 in dry season)
P.I.	do.	not to be sealed	4 to 9 (12 in dry areas)

Example :

Determine total pavement thickness required over a plastic subgrade whose C.B.R. is 5 per cent. Present 12 hour traffic count, 20 heavy trucks, 10 light trucks, 2 buses. Count made on a Thursday. Average annual rainfall 35 inches. Number of wet days per year 160. Life to be assumed as 20 years, traffic increasing at 5 per cent per annum. Pavement widths to be assumed as 16 ft for first 10 years, then increased to 20 feet.

Solution :

Present traffic $A=32$ trucks and buses $F=0.99$. Standard 5000 lb repetitions per 3 ft loaded strip for traffic in the first 10 years $=\frac{32}{0.99}\times 255\times 12.6\times 0.3=30{,}855$ or 31,000.

5000 lb repetitions per 3 ft loaded strip for traffic in second 10 years $\doteq\frac{32}{0.99}\times 255\times(33{\cdot}1-12.6)\times 0.21=36{,}000$.

Total repetitions should, therefore, not exceed 67,000.

Applying a climatic Factor of $\frac{35\times 160}{10000}$, i.e. 0.56, the number of repetitions is reduced to say 37,000, for which a soil with a C.B.R. of 5 per cent requires a pavement of 12 in. thick, Fig. 1.

9.2. Pavement thickness are also estimated from the formula,

$$T=\left(\frac{N}{100}\right)^{0\cdot 1}\left(\frac{R}{50}\right)^{0\cdot 3}\left\{16^3\sqrt{\frac{20}{CBR}}-8\right\}$$

where T = Thickness in inches

N = Average number of trucks per day (trucks and semi-trailers, commercial trailers and buses)

R = Average rainfall in inches, and

CBR = California Bearing Ratio at 95 per cent of the modified AASHO compaction.

Local conditions of drainage must be taken into account and where this is poor, where the land adjacent to the road is irrigated, or where the crown level is fixed, as in a street, the value of R should be taken as 50 in. where the actual rainfall is less than 50 inches.

Increased thickness should be used where the sub-soil is highly organic and where fine silts occur in poorly drained locations. Certain silty materials have high supporting value when well drained, but extremely low value if waterlogged.

9.3. Soil Moisture Relation (S.M.R.) Method

In New South Wales, the C.B.R. tests are not carried out. But the S.M.R. (soil moisture relation) rule or the Grading Rule is used to arrive at the thickness of pavement.

S.M.R. Rule

Let U = Upper Solid Limit = P.L. if plastic = L.L. if non-plastic.

S = Linear Shrinkage from L.L. of material passing No. 7 B.S. expressed as a percentage of original length.

W = Maximum dry weight (lb per cu. ft.) by Standard Proctor Compaction Test.

Effective cover required in inches is then —

$32+0.16U+0.27S-0.24W$ for heavy loading, and

$26+0.13U+0.22S-0.195W$ for normal loading.

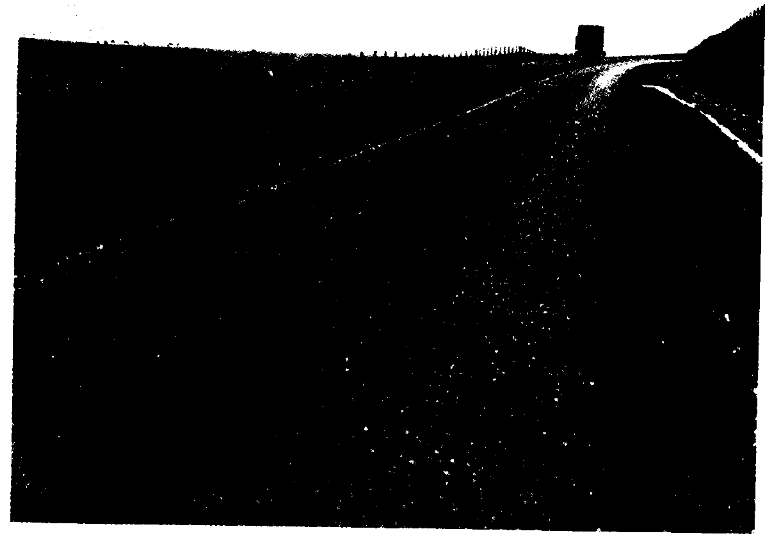


Photo 1. Typical finished non-skid surface

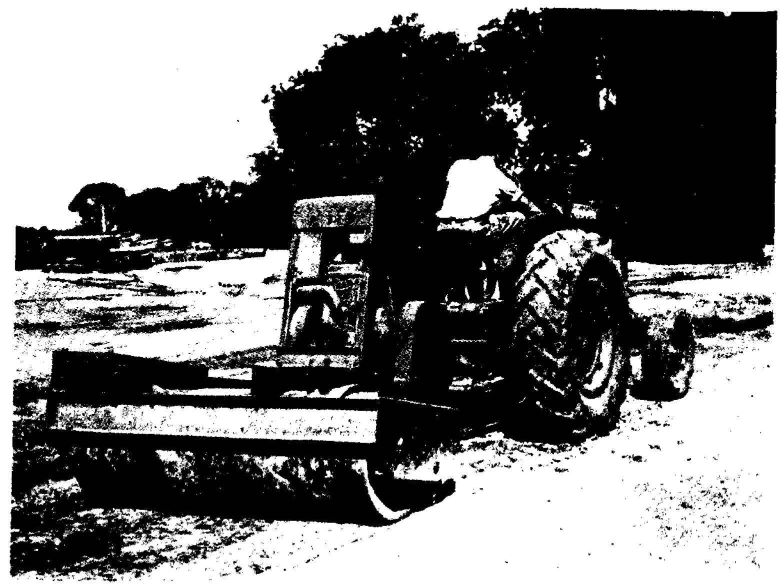


Photo 2. Vibrating Roller at work

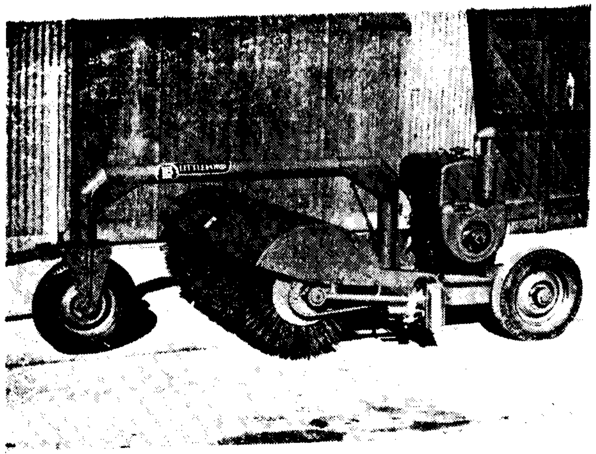


Photo 3.

Mechanical broom

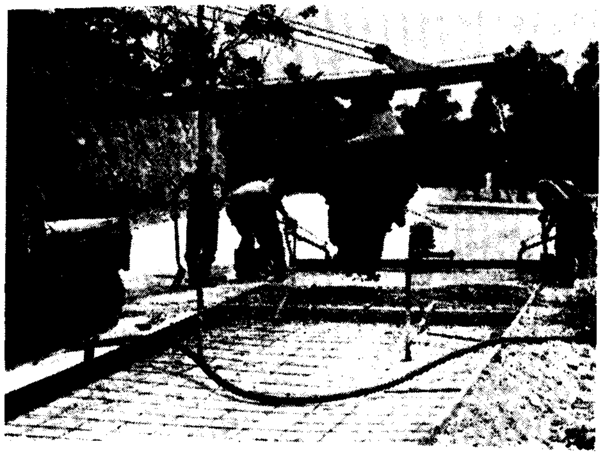


Photo 4.

Screeding of concrete

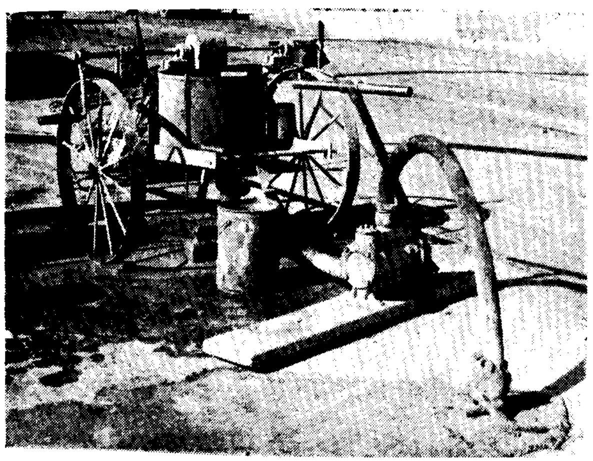


Photo 5.

Raising concrete pavement by mud-jacking process. General view of machine

Photo 6.

Pneumatic tyred roller at work

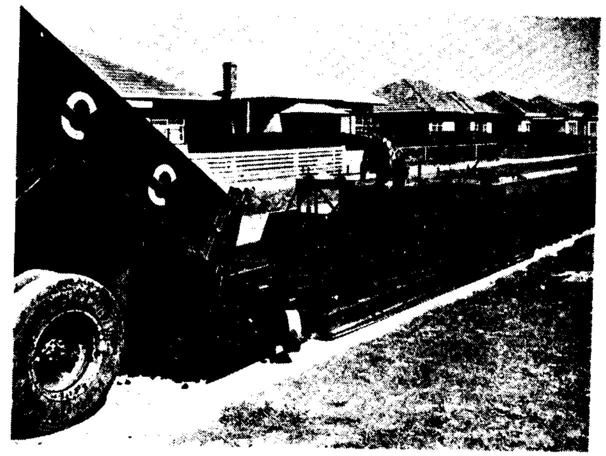


Photo 7.

Spreading plant mix

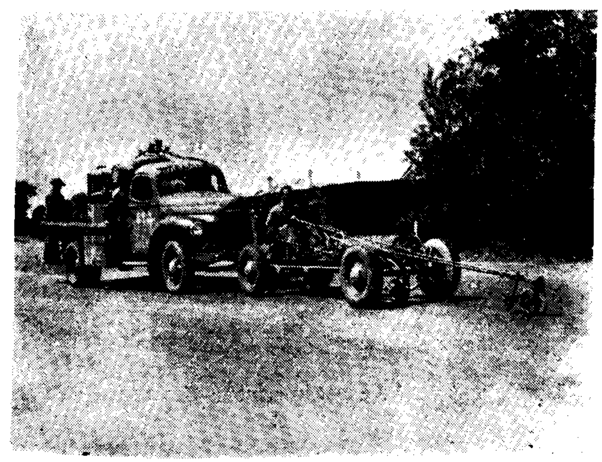


Photo 8.

Centre line marking machine

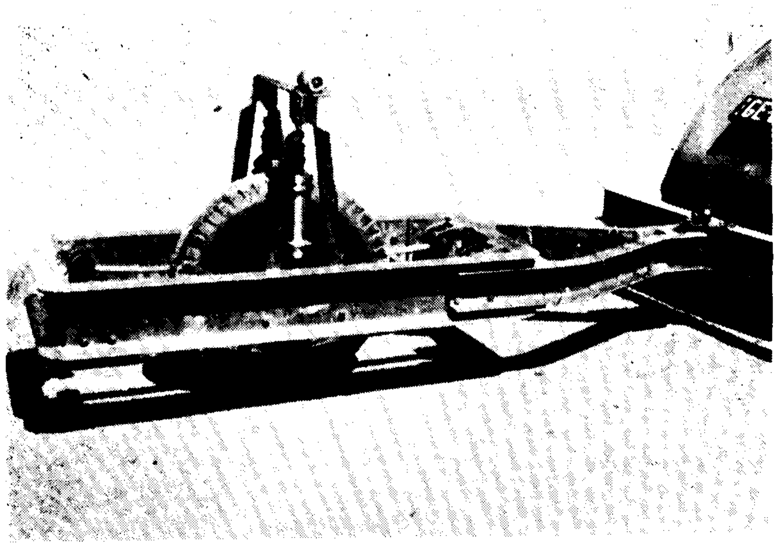


Photo 9. **Roughometer**

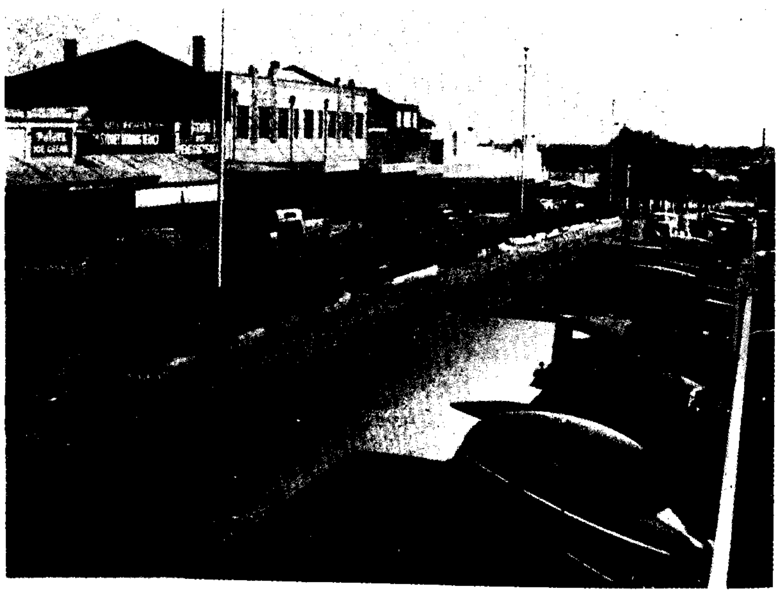


Photo 10. **Median strip in a main street—Leeton**

9.4. Grading Rule

Disregard all material retained $\frac{3}{4}$ in. square sieve. Compute the following ratios:

	Ratio (per cent) of all	To all Passing
R =	passing $\frac{3}{4}$ in. square sieve but retained on No. 7 British Standard Sieve	$\frac{3}{4}$ in. square
A =	passing No. 36 B.S.	No. 7 B.S.
B =	passing No. 200 B.S.	No. 36 B.S.
C =	less than 0.0135 mm	No. 200 B.S.

Let D, E and F be departures of A, B and C respectively outside range 40 to 60.

e.g. $D = A - 60$ if A is greater than 60.

$= 0$ if A is from 40 to 60.

$= 40 - A$ if A is less than 40.

If neither A, B nor C is less than 40, compute the sum of $D+E+F$.

If neither A nor B is less than 40, but C is, compute the same sum but count not more than 20 for F.

If A is not less than 40, but B is, compute the same sum but count not more than 20 for $E+F$.

If A is less than 40, compute the same sum if it is less than 20, otherwise count $D+E+F$ as 20.

To the sum so determined add one-half the plastic index and subtract one-quarter R. Call this final total "T".

Effective cover required in inches is then:

0.15 T for heavy loading (maximum wheel load 13,500 lb)

0.12 T for normal loading (maximum wheel load 9,000 lb)

During mechanical analysis, if the dispersion fails, and the ratios of A, B and C are not determined the grading rule cannot be applied. The S.M.R. Rule only is taken into account.

If the dispersion does not fail:

1. If the adverse constituents are present in considerable quantity, the grading rule does not apply and the S.M.R. Rule only is taken into account.
2. In A_3 Soils (passing 200 less than 10 per cent of Pass 7 or less than 15 per cent of Pass 36), free from appreciable adverse constituents, S.M.R. Rule does not apply, and grading rule is taken into account. (If much adverse constituents are present, S.M.R. Rule applies but not grading).
3. In other cases, average of the two rules is taken.

9.5. Gravel & Sand Clay to receive Surface Treatment

Requirements are for material as finally compacted in the pavement.

Materials will be suitable if it complies with the following requirements:

1. Total T not to exceed 5; (See Grading Rule)
2. Upper solid limit not to exceed 20; (See S.M.R. Rule)
3. Plastic limit not to exceed 8;
4. Maximum dry compressive strength not less than 400 lb per sq. in.

Oversize (retained in $\frac{3}{4}$ in. sq. sieve) not more than 5 per cent.

9.6. Gravel & Sand Clay for Use without surface Treatment

(See S.M.R. and Grading rules for symbols).

(a) For riding comfort oversize (retained $\frac{3}{4}$-in. sq. sieve) maximum 5 per cent.

(b) For easy maintenance and wear resistance (See Grading Rule for symbol R).

R over 40 per cent	Very good
R 20 – 40 per cent	Good
R 10 – 20 per cent	Less satisfactory
R under 10 per cent	Poor

(c) For stability against corrugation, rutting and ravelling.

Total T (substituting 65 to 60 in computing D, E and F) not to exceed 5. (See S.M.R. and Grading rules for symbols).

Neither A, B nor C to be less than 45.

Upper solid limit not more than ($20 \pm \frac{1}{4}$ P.I.)

Plastic index not to exceed 15.

Maximum dry compressive strength not less than 400 lb per sq. in. for the finally compacted pavement.

10. FLEXIBLE PAVEMENTS (NON-BITUMINOUS)

10.1. Where a road carries very little traffic, mere earth surface is found adequate in dry weather. But in locations where the road is called upon to carry traffic in wet weather, the surface is strengthened and stabilized with additional ingredients such as a better type of soil, bitumen or cement, so that the P. I. of the resulting mixture is not more than 5 and L. L. is not more than 25 if it is to be sealed later on. If not, these limits are raised to 10 and 35, respectively. A disc harrow and grader, or a Rotary Hoe or Siemen's Pulvi-mixer, is used for ensuring thorough admixture of the stabilizing materials. Sheepsfoot roller and wheeled rollers are used for consolidation at optimum moisture content. Bitumen and cement stabilization are not generally used for pavements, though they are employed for runways.

10.2. Gravel and crushed rock pavements are in general use particularly in the suburban districts. Gravel includes road-making sand also. A typical specification is given below.

	Percentage by Weight Gravel
Passing 1 in. screen (square openings)	100
Passing 3/16 in. screen (do.)	35 – 65
Passing No. 7 B. S. sieve (including elutriable)	27 – 54
Passing No. 14 B. S. sieve	22 – 44
Passing No. 36 B. S. sieve	15 – 35
Passing No. 200 B. S sieve	10 – 25

The material passing No. 36 B. S. sieve shall have lower liquid limit not more than 35, if to be sealed, L.L. not more than 25, plasticity index not more than 9. If it is not to be sealed, P. I. not more than 6. Tensile strength to be not less than 40 lb per sq. in.

10.3. Graders are employed to shape and maintain the gravel surface. It is remarkable that these gravel roads carry a tremendous amount of traffic of heavy loads without much deterioration. The reason appears to be that the load is really supported by the adequately designed pavement and sub-base, and gravel forms only a thin armour at top to protect the base and serves as wearing coat.

10.4. A special feature in the specification for the water-bound macadam pavement is the provision for two distinct sizes of aggregate to be applied in two layers, each to be laid separately, one to fill the voids in the other. No blinding or binding material is provided for in the New South Wales specification but the specifications of Victoria provide for very fine stone dust binder after the voids have been filled up by the screenings.

10.5. To have a dust free gravel road, a cheap process is sometimes employed. The gravel surface is well consolidated and swept. Then a thin layer of bitumen and oil at 0.15 gallon per sq. yd. is applied and allowed to soak in. Then it is covered with another layer of lighter oil at 0.1 gallon per sq. yd. and the surface covered with a layer of coarse sand at 1 cu. yd. per 150 sq. yd. This surface is expected to stand well for three years.

10.6. The traffic is almost entirely rubber-tyred. So a thin wearing surface is found sufficient. For bitumen surfacing work, different types of aggregates are in use, such as basalt, quartzite, and furnace slag Scoriala, kind of volcanic deposit, specially for bituminous seals.

An interesting instance was noted where over a water-bound macadam surface a coat of coke oven tar was applied at 0.2 gallon per sq. yd. and a layer of sea sand spread on top. The surface was expected to stand well for two years.

10.7. The finish of the road surface is specially attended to. The surface is checked both longitudinally and crosswise with a spirit level for level, cross slope or camber.

11. FLEXIBLE PAVEMENTS (BITUMINOUS)

11.1. Under this head, two types of work are included namely, surface treatments and asphaltic concrete work. The former comprises 'spray' work and the latter premixed work under 'initial treatment' or 'Re-seals'. One feature of the initial treatment is the invariable use of primers or tar, fluxed bitumen or cut-backs. Tar is used for dense fine-grained surfaces and bitumen for more open or absorbent surfaces.

11.2. Bitumen is invariably fluxed with asphaltic oil and power kerosene to bring the viscosity below 85 penetration. The spraying temperature ranges from 350 deg. (for pure bitumen) to 250 deg. (for cut-backs).

11.3. The aggregate used is usually ½ in. or ¾ in. gravel or fine crushed rock. The rate of application of aggregate is calculated at 21/32 of the average (least) dimension in inches. The aggregate is lifted from stockpiles, screened and loaded into trucks by mechanical aggregate loaders and spread over the bitumen layer by fan tail or cockerel spreaders attached to the rear of trucks or tractors.

11.4. The surface is rolled down with multi-wheeled pneumatic tyred rollers and 6-ton iron rollers, one pass with the former being followed by one pass with the latter. The multi-wheel roller is loaded with ballast to increase the load, and drawn by tractor or tractior (a tractor and air compressor combination which is a very useful unit). Traffic is kept off the road for 24 hours. The idea is to obtain a non-skid surface. Bleeding is not permitted. The asphalt is rarely visible on the surface.

11.5. It is significant that while 80/100 penetration bitumen, at the rate of 70 lb per 100 sq. ft. for semi-grouting and 40 lb per 100 sq. ft. for surface painting work, is used in India, only 28.3 lb per 100 sq. ft. (0.25 gallon per sq. yd.) as against 40 lb is used in Australia. For light pavements, the primer tar at 0.15 gallon per sq. yd. and bitumen binder at 0.2 gallon per sq. yd. (23 lb per 100 sq. ft.) and aggregate ⅜ in. size at 1 cu. ft. per 150 sq. ft., are used. Thus it will be seen that the Australian specification is much cheaper than ours. This specification may be not quite adequate for our conditions with a heavy rainfall and mixed traffic. However the possibility of considerable economy in our present usage of materials is indicated.

11.6. Where multi-coat seals are required, 2 or 3 applications of binder are made, each application being covered with smaller aggregates than that used in the previous layer and rolled.

11.7. Premixed work is done by the road-mix or drag-mix 'in situ' where the hot mixture of bitumen and screenings is dumped in the drag frame which is drawn over the road surface and the meterial spread to a uniform thickness and rolled down. In the case of plant mix, the mixture conveyed in trucks from a central depot where a 'Hot Mix Plant' produces the mixture to the desired proportions and consistency, is dumped on a 'Barber Greene Tamping and Levelling Finisher,' spread on the road and consolidated with pneumatic tyred and iron rollers.

11.8. Asphaltic concrete is not very much in use. It is being replaced by cement concrete wherever the traffic demands a hard pavement. In the existing asphalt concrete pavements, the base course consists usually of a layer of rolled concrete, of a lean proportion $1:2\frac{1}{2}:12$ with a water-cement ratio of 0.7 to form a firm base, the mortar being applied by grouting. Due to the difficulty in obtaining a good surface finish, this method is not used for concrete pavement work.

12. RIGID PAVEMENTS (CEMENT CONCRETE)

12.1. Concrete pavements are not much in vogue in Victoria, but they have been provided in the urban areas of New South Wales where there is heavy traffic. The uniform thickness is usually adopted. Light reinforcement in the form of a spot welded M.S. fabric with $\frac{1}{4}$ in. rods at 9 in. centres in one direction and 13 in. centres in the perpendicular direction is used, not so much to add to the flexural strength of the pavement, as to ensure structural integrity. Dowel bars are provided across the longitudinal and transverse joints; and they consist of $\frac{5}{8}$ in. rods 5 ft long, placed at 5 ft centres for the longitudinal joints and 2 ft centres for the transverse joints. In the latter case, one-half of the dowel bar is embedded in the slab, while the other half is capped with bitumen and cardboard capping. The aggregate used is $\frac{3}{4}$ in. and $1\frac{1}{2}$ in. size gravel, 50 per cent of each size being used in the proportion $1:2:3\frac{1}{2}$ for concrete. Concrete is usually supplied at site in transit-mixers mounted on trucks, from the Central Ready-mix Batching Plant. Mechanical vibrators are used for tamping and the top is finished by floats and by drawing a hessian cloth over the surface to obtain a non-skid surface.

12.2. An interesting item of repair work to concrete pavement was 'mud-jacking,' to raise up the slabs that had settled down. The process consisted of forcing under the slab a wet soil-cement mixture under pressure. The solid content of the mixture remained behind to support the slab. The equipment consisted of a hand-operated pug-mill in which the slurry of soil, cement and water is mixed, a manually operated double-acting sludge pump to force the slurry through 3-in. diameter holes bored through the slab and a compressor and drills. All joints and weak spots are caulked with hessian. The slurry consists of fine silty loam (100 per cent passing No. 7 sieve, 95 - 100 passing No. 18, 15 - 40 per cent passing No. 100, and 5 - 25 per cent passing No. 200).

Soil is mixed in the proportion of 1 of cement: 7 of soil for ordinary subgrades with the addition of water to form a thick cream. Slurry is pumped, the pressure being gradually increased to 30-40 lb per sq. in. (maximum), until the slab begins to move. Pumping is not continued for more than 2 in. lift at a time.

13. BRIDGES AND CULVERTS

13.1. Timber has been used on a large scale for road and railway bridge work in Australia. A plentiful supply of good timber and hard wood of the Eucalyptus and Messmate etc. varieties, which are not subject to white ant attack, has made the use of timber very economical and popular. The wood is seasoned and treated before use. Composite trusses of timber and steel are also largely in use. Recent constructions, however, are of steel or R.C.C. These are generally of R.C. piles and cross heads with R.S. or R.C. girders and a concrete slab deck. It is a common practice to mould the R.C. members except the deck slab in the precasting yard and convey them to site and fit them up in position. For want of enough skilled masons, the abutments are also made of the R. C. retaining wall type moulded 'in situ'.

13.2. The loading used is $H_{20}S_{16-44}$ of the American Association of State Highway Officials, which is about 20 per cent lighter than the I.R.C. Class A loading. The Table below shows details of the amended loading together with details of the old Country Roads Board (Victoria) Class A loading for Comparison:

Class of Bridge	Class of loading	Unif. load per linear ft per lane in lb (9 ft width)	+ Knife load placed to give max. stress in lb per ft of lane (9 ft)	
			For Moment	For Shear
A	H — 15	480	13,500	19,500
B	H — 10	320	9,000	13,000
AA or revised AASHO	$H_{20}S_{16-44}$	640	18,000	26,000

13.3. The majority of bridges in Victoria have spans of 30 to 60 ft and the creeks over which they are constructed are narrow and shallow compared to Indian streams and rivers. Gradients and curves on long bridges are quite common. A minimum of 22 ft width is provided for bridge decks on the Main Highways and 20 ft on other roads. For culverts the distance between parapets is equal to the full formation width.

13.4. Distribution of Concentrated Loads on Slabs

In the direction perpendicular to the span of the slab, the wheel load are considered as distributed uniformly over a width of slab known as 'effective width' and is determined by the following equations, where S = span of the slab in feet, W = width of wheel in feet, x = distance in feet from centre of nearest support to centre of wheel, and E = effective width in feet for one wheel.

Case I. Main Reinforcement Parallel to Direction of Traffic

E shall not exceed 7 ft for a single wheel. E is given by $E = 0.7\,S + W$. When two wheels are so located on a transverse element of the slab that their effective width will overlap, the effective width for each slab shall be $\frac{1}{2}\,(E + a)$ where a is the distance between centres of wheels.

Case II. Main Reinforcement Perpendicular to the Direction of Traffic

$E = 0.7\,(2x + W)$. The Bending Moment on a strip of slab 1 ft wide shall be determined by placing the wheel loads in the position to produce maximum Bending Moment, determining the effective width for each wheel, and assuming that the load delivered by each wheel to the 1 ft strip to be equal to the wheel load divided by its respective effective width.

Also $E = 0.7\,S + W = \sqrt{[(a + 2d)^2 + D^2]}$ for slab spans parallel to direction of load, and

$E = 0.7\,(2x + W)\sqrt{[(b + 2d)^2 + D^2]}$ for slab spans perpendicular to direction of load, where

a = width of wheel tyre contact in the direction of the slab span

b = width of wheel tyre at right angles to slab span

d = depth of filling or cushion, and

D = slab thickness,

13.5. R. C. precast channel section slabs

These are largely used for culverts of 4 to 20 ft spans in Victoria. The reasons for using the precast units are :

(1) difficulty in obtaining sufficient skilled labour on site of work ; and

(2) simplicity of manufacture of such units in a central casting yard ;

Precast slabs for culverts up to a clear span of 10 ft and precast beams for spans up to 22 ft are used.

The deck consists of a number of such units placed adjacent to each other along the span of the culvert. The flanges bear at their ends on the abutments and support the web which forms the deck proper. A typical section is shown in Fig. 2.

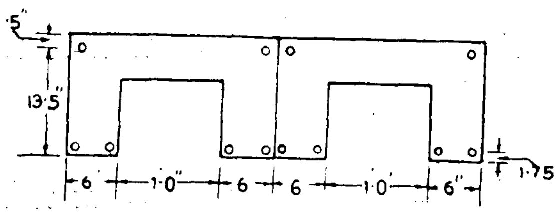


Fig. 2.

13.6. Proportioning Concrete

In New South Wales, the specification for R. C. bridges classifies the concrete into different categories with strength at 28 days as given below :

Class AA	Concrete	1 : 1½ : 3	Strength	3000	lb per sq. in.
Class A	,,	1 : 2 : 4	,,	2500	do.
Class B	,,	1 : 2½ : 5	,,	2000	do.
Class C	,,	1 : 3 : 6	,,	1500	do.

Concrete of Class AA and Class A is used for precast footway slabs, heavy sections, deck slabs, thin vertical sections and columns and thin confined horizontal sections, while class B and C are used in locations where the concrete has not to take up any load like head walls and inlet sumps.

Bridges with composite trusses of timber and steel are also very popular. Several of the old bridges are of this type. The timber bridges are standing very well with proper and adequate maintenance, even though they carry very heavy and constant traffic.

The Sydney Harbour Bridge—a steel structure of 1,650 ft, is the largest single span rigid arch in the world. The width of the deck is 160 feet. The bridge is designed for 6 lanes of traffic in the middle, with 2 lanes of electric railway on one side and 2 lanes of tramway on the other, and a foot walk beyond, on each side. Out of these 6 lanes of traffic, 4 lanes of traffic are allowed for city bound vehicles in the morning and out-bound vehicles in the evening. The bridge has consumed 50,000 tons of steel and it took 9 years to build at a cost of £ 9.75 million, out of which £ 8 million have been met from loan funds. The total expenditure per day for maintenance and loan charges amounts to £ 1,000 and this sum is obtained from tolls levied on the bridge. The bridge forms the most prominent feature on Sydney's skyline.

14. HIGHWAY MAINTENANCE

14.1. Maintenance work is carried out by road units, each consisting of a patrolman and two assistants and a cu. yd. tipping truck. Patchwork is done with bitumen premix using about a gallon per sq. yd. and 3/8 in. screenings. Gravel is conveyed from the quarries and spreading and grading is done with a light rubber tyred grader. The grader used with truck patrols is primarily meant for grading of shoulders and after rains it is used for light blading of unsealed sand and gravel pavements. One such unit looks after a stretch of 60 miles of road.

14.2. Roughometer

For ascertaining the smoothness of the surface on a pavement, a machine called the Roughometer is employed by the Country Roads Board, Victoria. It registers the bumps on the road for every mile and gives an idea of the comparative riding qualities of the road.

The instrument consists of a heavy frame which is carried on two single leaf springs, which carry the axle of a standard motor car wheel, Photo 9. Dashpots are provided as a damping system to control spring oscillation and vertical movement of the axle relative to the frame is integrated by a mechanism consisting of two opposing back clutches. For each inch of integrated

movement, an electrical impulse is sent to the electrical counter which thus records the vertical movement in inches. The vibration of the frame produces an electric contact every time it moves, and this is transmitted by a cable to a counter which records the number of vibrations. Driving at 20 m.p.h., the counting is done every 3 minutes, i.e. every mile. This gives a comparative idea of the roughness of the surface. The inflation pressure should be 30 ± 0.5 lb per sq. in. A speed of $20 \pm \frac{1}{2}$ mile per hour is found to be the most suitable.

15. MEDIAN STRIPS

15.1. In wide avenues, particularly in the urban areas, separate median strips are provided to demarcate the line of separation of the lanes for incoming and outgoing traffic, Photo 10. These take the form of a curb 2 to 4 ft wide with concrete edging and grass turfing in the middle. Sometimes the turf is replaced by beautiful flowering plants as in the Anzac Avenue, Sydney. In certain cases, the median strip is made fairly wide and serves as a parking space for cars. In Sydney there are also avenue trees planted at intervals. On the concrete kerbs on both sides of the median strips, spherical glass reflectors are embedded into notches made on the edges of the kerb, for the guidance of drivers at night.

16. TRAFFIC CONTROL

16.1. Traffic control is a part of the Engineer's duties. Each Division has a traffic inspector who checks overloading on the trucks by using the loadometer and reports cases of overloading to the Magistrates for punishment.

16.2. Restriction of Loading on Roads

In Victoria, the following are the principal provisions of the Law regarding the dimensions and weights of vehicles, but provision is made for granting special permits :

(a) The gross weight on any one tyre not to exceed 5,000 lb.

(b) The gross weight carried on any one axle not to exceed 17,000 lb.

(c) The gross weight on all axles or any consecutive group of axles not to exceed the weight shown in the Schedule varying between 11 tons 16 cwt and 26 tons 1 cwt having regard to the distance between the extreme axles of the vehicle or of a group of axles.

(d) Tyre pressure not to exceed 100 lb per sq. in.

(e) Width of vehicle and load not to exceed 8 feet.

(f) The overall length of vehicle other than articulated vehicles not to exceed :

(1) in the case of a vehicle used for carrying of passengers for hire...33 ft

(2) In any other case......31 ft

(g) Overall length of an articulated vehicle not to exceed 45 ft length ; the length of either of the rigid parts not to exceed 35 ft.

(h) Overall length of vehicle and trailer not to exceed 50 ft.

The speeds range between 12 m. p. h. for a vehicle with a gross weight exceeding 3 tons and fitted with one or more tyres other than pneumatic tyres and 40 miles per hour in the case of a pneumatic-tyred passenger carrying vehicle or a goods vehicle, the gross weight of which exceeds 50 cwt but not 3 tons.

16.3. The traffic on main roads is controlled at junctions by automatic light signals of four tyres : (1) triple light type, (2) dial type, (3) the horizontal tube light type, and (4) the traffic actuated type.

16.4. The incidence of traffic accidents is very low indeed considering the high speed permitted. This is mainly due to the absence of pedestrians or stray animals on the roads and the keen road sense of the drivers and pedestrians alike.

16.5. Traffic lanes are marked on all the main roads and double full lines are marked where overtaking is restricted. Four-lane pavements are invariably provided with median strips fitted with glass cats-eye reflectors. Road signs and advance signs are provided systematically and painted with lacquer and glass beads of the size of fine sand to increase visibility.

17. CONCLUSIONS

17.1. The one impression about Australian Highway technique that is outstanding is the very close attention that is being

paid to the collection of statistics and estimation of traffic trends in a systematic manner and their application to road design. Testing of the subgrade soils is invariably done either in the field or at the central laboratories, and the road structure is designed to suit the traffic that the road is expected to carry. If the soil is weak, its bearing capacity is increased by stabilization with addition of suitable materials wherever necessary. Because of the careful design, even roads with a light seal of gravel or bitumen carry heavy loads without deterioration.

17.2. The traffic on the roads is almost entirely rubber-tyred. So there is a preference for flexible pavements. Cement is in very short supply, whereas there is not much difficulty in obtaining supplies of bitumen. In Victoria cement concrete is sometimes used as a base course for a bitumen-wearing surface and rarely for the finished pavement, but in Sydney and suburbs concrete pavement is adopted on a large scale. A light reinforcement is usually used more to afford proper distribution of the load and minimise cracking than to support the load.

17.3. A serious problem in the development of highways, as in all other spheres of activity, is the shortage of manual labour. Labour is difficult to obtain and is costly. The basic wage of an unskilled labourer is over £ 10 for a week of 40 hours. But at present there is a regular inflow of labour from European countries like Greece, Czechoslovakia, Hungary and Italy. The immigrant labourers (called New Australians) are obliged to work on Government projects for a definite period after landing in Australia. Thereafter they are free to choose their careers. The labourers are provided with every convenience regarding food and accommodation. Special camps with tents or huts, kitchens and dining halls, recreation rooms, baths and washing facilities, etc., are provided at all important workspots. Very good food is given at fairly cheap rates. Cots, mattresses and blankets are also supplied by the Department. As a result, there is willing co-operation between the labourers and the supervising staff.

17.4. The difficulty of obtaining adequate manual labour has resulted in the mechanisation of all the processes, wherever possible. For earth moving and clearing, bull-dozers, rippers, scrapers and scoops are used. For quarrying and transport of gravel, bull-dozers, mechanical shovels, aggregate loaders and tipping trucks are employed. Rolling is done with iron-wheeled or vibratory rollers and sheepsfoot or pneumatic tyred rollers drawn by tractors. A bitumen surfacing equipment is a large train of vehicles consisting of bitumen boiler and pumps, tank wagons, sprayers and drag-spreaders and plant mix equipment of various types. To keep the several items of machinery properly serviced, a well-equipped central workshop is maintained with a liberal stock of spares in a Central Store. A small workshop and laboratory is available in each division. A mobile workshop extends repair facilities to out-of-the-way localities also. Mechanisation has resulted in general speeding up and standardising of the work.

17.5. The design for important bridges are made in the Central Offices. In Victoria, the execution is also usually carried out by the Bridge Division attached to the Head Office, but in New South Wales the execution is ordinarily done by the local Divisional Engineers.

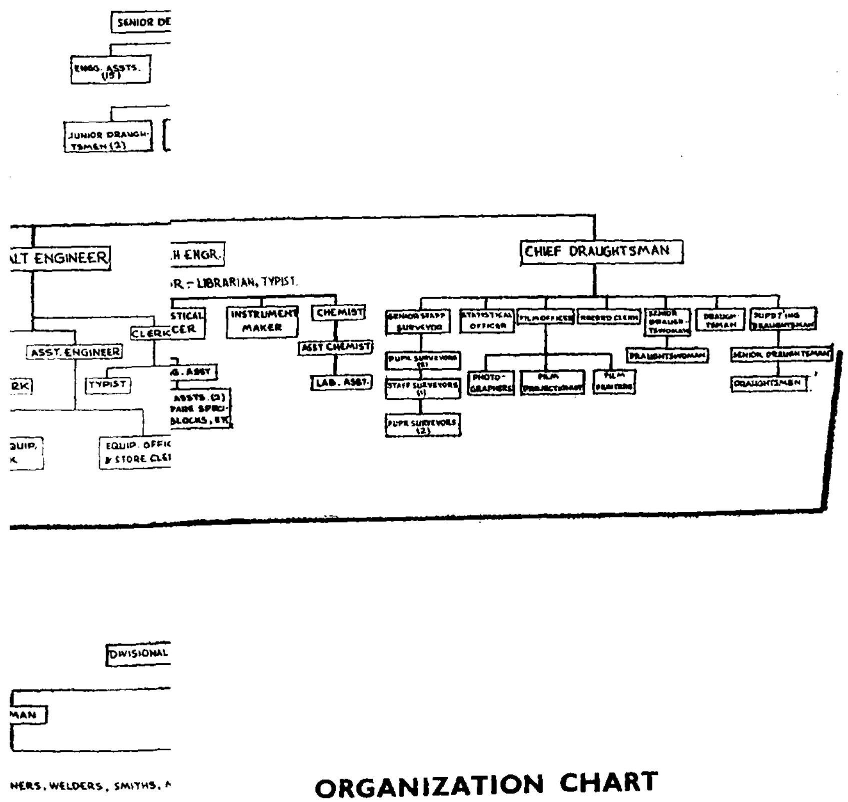
17.6. Another noticeable feature was the efficient control of traffic on the highways. Roads are designed with sufficient width and horizontal and vertical curvature to meet the demands of the volume and speed of traffic expected to use them. The drivers have got a keen road sense and it is surprising that accidents are so rare when we consider that the usual speed of motor vehicles is about 60 miles per hour. There is rarely any overloading of commercial vehicles.

17.7. Problems in India are very different from those in Australia. In India, labour is comparatively cheap and plentiful and work has to be found for them. So the idea of mechanisation on a large scale, as in Australia, has to wait. Also, traffic conditions are different. Our roads have to provide for a mixed traffic consisting of the slow-moving iron-tyred bullock carts and motor vehicles.

17.8. Collection of statistics about traffic volumes on all roads, estimates of future traffic and application of the results to road design, adequate geometric standards and programmed execution so that work done in the immediate future forms part of a properly designed final plan are important.

Photos 1, 2, 6 to 9 are reproduced with acknowledgments to the Country Roads Board, Victoria; Photos 3 to 5 and 10 with acknowledgments to the Department of Main Roads, New South Wales (Australia).

PLATE I



ORGANIZATION CHART COUNTRY ROADS BOARD, VICTORIA (AUSTRALIA)

THE PLASTIC THEORY AS APPLIED TO THE DESIGN OF MILD STEEL BEAMS AND FRAMES

By

MISS SHAKUNTLA S. JOSHI, B. E.

1. Study of the behaviour of mild steel beyond the yield point when tested under axial loads i.e. in tension or compression or under lateral loads i.e. in flexure is necessary for the correct understanding of the plastic theory in the design of mild steel beams and frames. Appendix 1 gives in tabular form the details of the experimental or theoretical stress-strain relations established by different workers in the field. The corresponding diagrams of stress distribution in flexure in elastic, partially plastic and fully plastic range are also given. It is seen that some workers like Ewing, Muir and Binnie did not get any drop in stress at yield point, or they did not attach any significance to what they got, while some others, especially Robertson and Cook got a well defined drop in stress at yield because of the special device used to ensure axial loading.

The stress distribution given by Robertson and Cook has been generally accepted, Fig. 1.

2. Assumptions

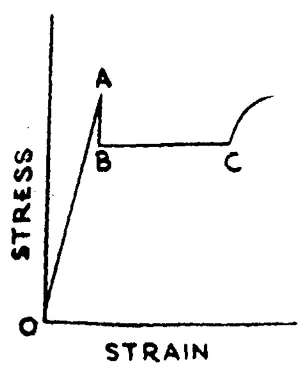


Fig. 1

The elastic theory is based upon the followiug assumptions:

(i) Upper yield point A is ignored. Plastic range starts from B and ends at C.

(ii) Stress-strain relationship for compression is identical with that for tension.

(iii) Plane sections remain plane after bending.

3. Stress Distribution across a Section under an increasing load

Consider a beam of prismatic section, simply supported at its ends and subjected to pure bending. In the initial stages of loading, when the stress in the extreme fibres has not reached the elastic limit, the stress at any point in the cross section will be proportional to its distance from the neutral axis. This is the stress distribution according to the elastic theory of bending, Fig. 2.

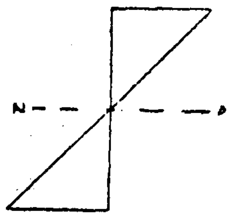


Fig. 2

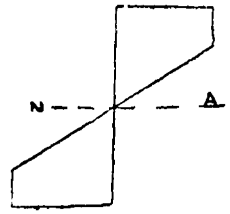


Fig. 3

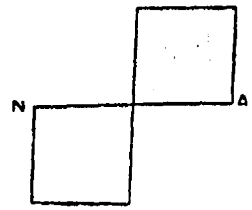


Fig. 4

When the load is increased until the extreme fibres are stressed to the yield point, the stress is still proportional to the distance from the neutral axis. If the load is increased still further, the extreme fibre offers no more resistance and the fibres in the inner layers get stressed to the yield point. The section is now partially plastic and the distribution of stress is as shown in Fig. 3.

If the load is still further increased, the section becomes fully plastic, Fig. 4. Any attempt to increase load further will result in increased deflection with no further increase in the moment of resistance. The section now acts as a hinge or a 'plastic hinge' is said to be formed at the section and ultimately the beam collapses.

4. Definition of Terms

(1) **Plastic Hinge Moment.** The bending moment of a section when a plastic hinge is formed is called the plastic hinge moment. In the case of bending without axial load, it is equal to the plastic moment of resistance.

(2) **Collapse Load.** (c) The collapse load is that load which when placed on a structure will produce sufficient number of plastic hinges in the structure to cause failure in the manner of a hinged mechanism. The word structure means either the whole structure or any part of it which can fail independently.

(3) **Plastic Moment of Resistance** (P. M. R.). This is the moment of resistance of a section when each fibre in the section has been stressed to the yield point. Fig. 5.

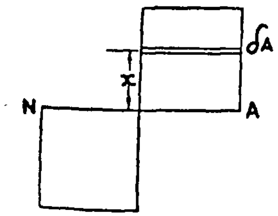


Fig. 5

Plastic moment of resistance $= \Sigma fy\, \delta A \times x$

$$= fy\, \Sigma \delta A \times x$$

$$= fy \times Z_p.$$

(4) Plastic modulus Z_p of a section is the sum first moment of a areas on the tension and compression sides of the section about the neutral axis.

(5) **Shape Factor 'V'.** The ratio of the plastic section modulus (Z_p) to the elastic section modulus (Z_e) is called the shape factor.

$$V = \frac{Z_p}{Z_e}$$

In the case of a rectangular section $b \times d$

$$Z_p = 2 \times b \times \frac{d}{2} \times \frac{d}{4} = \frac{bd^2}{4}$$

$$Z_e = \frac{bd^2}{6}$$

$$V = \frac{bd^2}{4} \times \frac{6}{bd^2} = 1.5.$$

The value of this factor for most standard rolled steel joist sections is about 1.15.

(6) **Load Factor** (q). The ratio of the collapse load (c) to the working load (W) is defined as the 'load factor'.

5. Hinge

Fig. 6 gives the relation between the bending moment and curvature in structural members. It is seen that any increase in curvature is accompanied by increase in bending moment, until the latter attains its full plastic value. Also, as the curvature increases indefinitely, bending moment tends to a limiting value, M_p, the full plastic movement regardless of the previous history of loading.

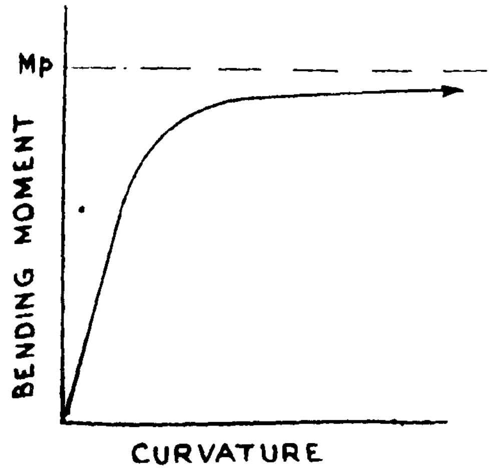


Fig. 6

When at a section, fully plastic moment is attained, the curvature becomes indefinitely large, so that a change of slope can take place over a short length of the member. The member behaves like a hinge at this cross section, the rotation of the hinge being possible when a resisting moment equal to 'M_p' is developed. This conception was introduced by Maier-Leibnitz.

6. If loads acting on a frame are increased, then the sucessive increasing bending moments are resisted firstly by elastic action and then by plastic action. Afterwards a plastic hinge will be formed at the highly stressed section. If loads are further increased, this hinge will rotate under M_p, and more plastic hinges will be formed in other parts of the structure and they will also rotate. Finally sufficient number of hinges will be formed to transform the structure into a mechanism which will then deform to an indefinite extent until its geometry changes appreciably resulting in a collapse.

7. The Mechanism of Failure

Designs based on the plastic theory are worked out for the collapse load. A knowledge of the position of the plastic hinges under the collapse load is essential before the plastic moment can be worked out. A simple illustration of the method as applied to beams is given below.

Consider a beam, Fig. 7, fixed at both ends and subjected to a uniformly distributed load. In the elastic

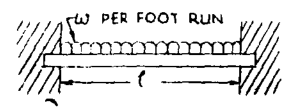


Fig. 7

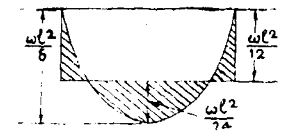


Fig. 8

range the bending moment diagram will be as shown in Fig. 8. As the load is gradually increased, the fixing moments tend to increase until the yield point is reached at the supports. At this stage "plastic hinges" are formed at the supports, and any further increase in the loading causes an increase in the moment at the centre without affecting the moments at the ends. With still further increase in load a hinge is formed at the centre also and then the beam fails, Fig. 9. At the time of collapse the bending moments at the supports and at the centre will each be equal to the plastic moment of resistance and hence the bending moment diagram will be as shown in Fig. 9.

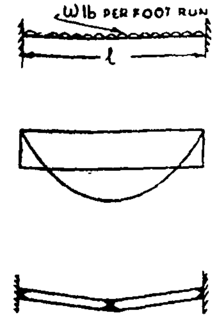


Fig. 9

The collapse moment diagrams showing hinges at the time of failure for some typical cases are shown in Fig. 10.

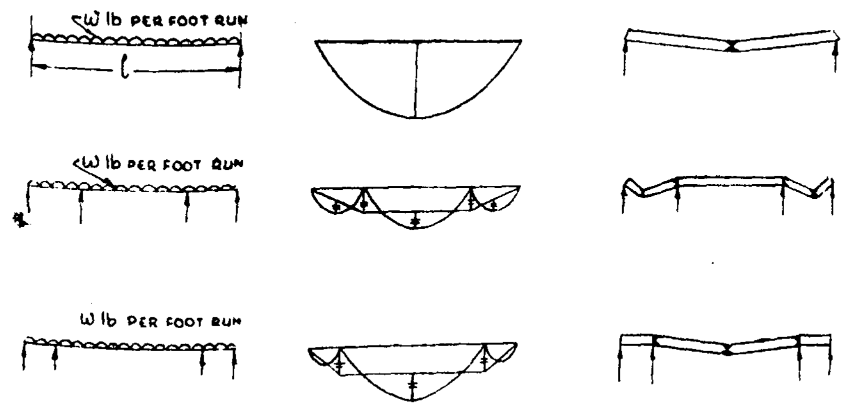


Fig. 10

8. The following Table, Fig 11, gives Mr (moment of resistance) according to elastic theory and P. M. R. (plastic moment of resistance) for some typical sections.

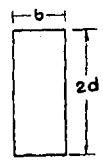
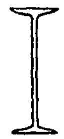
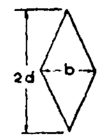
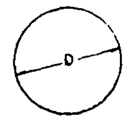
SECTION				
Mr	$\frac{1}{2}\times 10\times bd\times\frac{2}{3}\times 2d$ $=10\times\frac{2}{3}\times bd^2$	$=10\times Ze$	$=10\times\frac{bd^2}{6}$	$=10\times\frac{\pi D^3}{32}$
P M R	$15\cdot 25\times bd\times 2d$ $=15\cdot 25\, bd^2$	$=15\cdot 25\, Zp$	$15\cdot 25\times\frac{bd}{2}\times\frac{2}{3}d$ $=15\cdot 25\times\frac{bd^2}{3}$	$15\cdot 25\times\frac{\pi D^2}{8}\times\frac{4D}{3\pi}$ $=15\cdot 25\times\frac{D^3}{6}$
SAFETY MARGIN $\frac{PMR}{Mr}$	$1\cdot 525\times 1\cdot 5$ $=2\cdot 2875$	$1\cdot 525\times\frac{Zp}{Ze}$ $=1\cdot 525\times 1\cdot 15$ $=1\cdot 75$	$=1\cdot 525\times 2$ $=3\cdot 05$	$=1\cdot 525\times\frac{1}{6}\times\frac{32}{\pi}$ $=2\cdot 575$

Fig. 11

Assume permissible working stress according to the elastic theory = 10 tons per sq. in., yield stress = 15.25 tons per sq. in., breaking strength = 28 tons per sq. in.

Although the breaking strength of the material is 28 tons per sq. in., the margin of safety is not $28/10 = 2{\cdot}8$, because the section cannot resist bending moments in excess of its plastic moment of resistance which, therefore, is its maximum collapse moment or hinge moment. From Fig. 11, it is clear that the margin of safety under the conventional design methods has a very wide variation between 3 to 1.75. It is certainly not rational to design structures with such a wide variation in the margin of safety. While there is waste of material in designs which give higher values up to 3 for the safety margin, it is safe to design members with a safety margin of 1.75.

This then is the justification for introducing plastic methods of design. According to this method the desired margin of safety is first decided and is called the load factor. The working loads are multiplied by this factor to get the collapse load. The correct section is then so found that the beam collapses at that load.

In I-beams designed according to the elastic theory, it is found that the safety margin works out to be about 1.75. This value is, therefore, assumed as the load factor for the design of other sections also. The other consideration governing the design is the deflection which has to be controlled according to requirements.

9. The economy resulting from the plastic method of design is illustrated by a few examples.

Example I

Design a mild steel beam of circular cross section fixed at both ends, and carrying a central concentrated load of 5 tons. The span of the beam is 10 feet.

Plastic Theory. The load factor commonly used in designs by the plastic theory for mild steel is 1.75. Assume the same load factor in this case also.

Collapse load $= 5\times 1.75 = 8.75$ tons.

$$M_p = \frac{q\,Wl}{8} = \frac{8.75\times 10}{8}\times 12 = 131.25 \text{ in.-tons.}$$

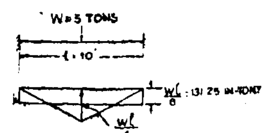


Fig. 12

Assume the stress at yield point as 15.25 tons per sq. in.

$$\therefore Z_p = \frac{131.25}{15.25} = 8.6 = \frac{D^3}{6}$$

(First moment of area about neutral axis.)

$D^3 = 6\times 8.6 = 51.6$ in.^{3}

$D = 3.72$ in.

Elastic Theory

Maximum $B.M. = \frac{5\times 10}{8}\times 12 = 75$ in.-ton.

Assume a working stress of 10 tons per sq. in.

$$Z_e = \frac{75}{10} = 7.5 = \frac{D^3\pi}{32}$$

$\pi D^3 = 7.5\times 32 = 240.0$ in.^{3}

$D = 4.25$ in.

Percentage saving $\dfrac{4.25^2 - 3.72^2}{4.25^2} \times 100 = 22.9$ per cent.

Example 2

Design a beam of I-section fixed at both ends, span 15 ft and carrying a uniformly distributed load of one ton per foot run

Elastic Theory

$$B.M. = \frac{Wl}{12} = \frac{1 \times 15 \times 15 \times 12}{12} = 225 \text{ in.-ton.}$$

$$Z_e = \frac{225}{10} = 22.5 \text{ in.}^3$$

Provide 10 in. $\times$ $4\frac{1}{2}$ in. $\times$ 25 lb I-section, giving $Z_e = 24.47$ in.^{3}

Total weight $= 25 \times 15 = 375$ lb

Plastic Theory. Load factor $= 1.75$.

$\therefore$ Collapse load $= 1.75$ ton per running foot. When plastic hinges are formed at both the supports and at the centre the moment at the centre is equal to the moment at the support.

$$\therefore \text{ Moment at collapse} = \frac{wl^2}{8 \times 2} = \frac{wl^2}{16}$$

$$= \frac{1.75 \times 15 \times 15}{16} \times 12 \text{ in.-ton.}$$

$$= 295.31 \text{ in.-ton.}$$

$$\therefore Z_p = \frac{295.31}{15.25} = 19.35 \text{ in.}^3$$

$$Z_e = \frac{Z_p}{\text{safety factor}} = \frac{19.35}{1.15} = 16.8 \text{ in.}^3.$$

Use a 9 in. $\times$ 4 in. $\times$ 21 lb I-Section giving $Z_e = 18.03$ in.^{3}

Total weight $= 21 \times 15 = 315$ lb.

$$\text{Saving} = \frac{(375 - 315) \times 100}{375} = 16 \text{ per cent.}$$

10. Procedure for design of single span fixed beams

(a) Multiply working loads which are acting simultaneously by the load factor to get the appropriate collapse loads.

(b) Draw the free moment diagram for the collapse load.

(c) Draw a line across this diagram to represent fixed end moments. Finally, this line serves as the base line from which actual collapse moments are measured. This line is called ' fixing moment line '.

(d) Note how many plastic hinges will be necessary to enable the beam to collapse. Adjust the fixing moment line in such a way as to give the values of maximum moments at the points where plastic hinges are required equal. Successive plastic hinge moments must lie on opposite sides of the base line.

(e) Calculate the plastic hinge moment and divide it by the yield stress, 15.25 tons per sq. in., to get the plastic modulus Z_p.

(f) Divide Z_p by the shape factor to get the required section modulus Z_e.

11. Continuous Beams

Example 3

Design a continuous beam $ABCD$ simply supported at the ends A and D and continuous over the intermediate supports B and C, carrying a uniformly distributed load of 3 tons per R. ft. $AB = 20$ ft, $BC = 30$ ft, and $CD = 26$ ft, Fig. 12.

Load factor $= 1.75$.

$\therefore$ Collapse load $= 3 \times 1.75 = 5.25$ tons per foot run.

Maximum free $B.M.$ in span $AB = \dfrac{5.25 \times 20 \times 20}{8}$

$= 262.5$ ft-ton

-do- -do- span $BC = \dfrac{5.25 \times 30 \times 30}{8}$

$= 590.6$ ft-ton

-do- -do- span $CD = \dfrac{5.25 \times 26 \times 26}{8}$

$= 443.6$ ft-ton

The bending moment diagram for the beam is shown in Fig. 12.

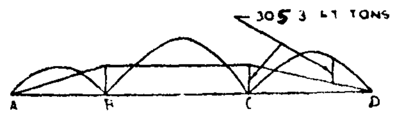


Fig. 13

For collapse of span CD, the maximum positive bending moment in span must be equal to the maximum negative bending moment at support C. It can be shown mathematically that, for a uniformly distributed load, each of these moments will be numerically equal to 0.688 the free bending moment in span.

$\therefore M_p = 0.688 \times 443.6 = 305$ ft tons. Considering span BC alone, collapse occurs when the moment in the span and at B and C are each equal to $\frac{590.6}{2} = 295.3$ ft tons.

The free bending moment in span AB, being the smallest of the three moments need not be considered.

Provide for the maximum plastic moment of 305 ft tons.

$$Z_p = \frac{305 \times 12}{15.25} = 240 \text{ in.}^3$$

Adopting R. S. J. (shape factor 1.15)

$$\therefore Z_e = \frac{240}{1.15} = 209 \text{ in.}^3$$

Use 24 in. $\times$ $7\frac{1}{2}$ in. $\times$ 95 lb R.S.J.

giving $Z_e = 211$ in.^{3}

Example 4

Design with non-uniform section.

For collapse in span AB, $M_p = 0.688 \times 262.5$

$= 180$ ft-ton

$$Z_p = \frac{180}{15.25} \times 12 = 141.6 \text{ in.}^3$$

$$Z_e = \frac{141.6}{1.15} = 123.2 \text{ in.}^3$$

Use 18 in. $\times$ 7 in. $\times$ 75 lb I-section, giving

$Z_e = 127.91$ in.^{3}

P.M.R provided = 186.6 ft-ton.

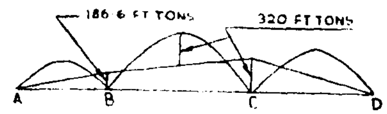


Fig. 14

Draw a fixing moment line from zero at A to 186.6 at B and to a value at C equal to maximum moment in span BC, 320 ft-ton, Fig. 13.

P.M.R. of reinforcement = 320 − 186.6

= 133.4 ft-ton

2 plates 9 in. $\times$ 3/8 in. P.M.R. = 133.1 ft-ton.

12. Summary of Procedure for the Design of Continuous Beams

(1) For Uniform Section

Select the end span and the internal span which carry the largest bending moment. Draw a line across the free moment diagram of the end span with zero B.M. at the free end so that the B.M. at the other end is equal to the maximum resulting moment in the span. Draw another line across the internal span so as to make both support moments equal to maximum moment in span. Use greater of the plastic hinge moments so obtained.

(2) For Non-uniform Section

Select the span which has the smallest bending moment and determine the plastic hinge moment in the way described above. Extend the fixing moment diagram across the remaining spans to find the plastic hinge moments required for these spans. Select a section to suit the smaller plastic hinge moment and reinforce it to carry the larger ones.

The plastic modulus Z_p of a pair of flange plates is equal to the cross-sectional area of one plate multiplied by the distance between their centres of gravity.

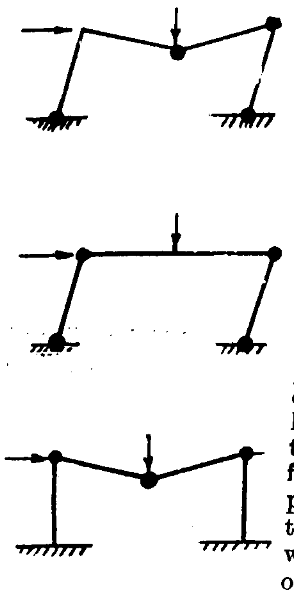


Fig. 15

13. The plastic design has its greatest use, with consequent economy, in redundant frames with fully rigid connections. The possible ways of failure of a porta lframe fixed at both ends is shown in Fig. 15. The following extract from B.S.S. 449 of 1948 is relevant.

"This method, as compared with the methods for simple and semi-rigid design, will give the greatest rigidity and economy in the weight of steel used, when applied in appropriate cases. For the purpose of such design accurate methods of structural analysis shall be employed leading to a load factor of 2, based on the calculated or otherwise ascertained failure load of the structure or any of its parts, and due regard shall be paid to the accompanying deformations under working loads, so that deflections and other movements are not excessive."

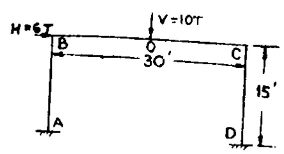


Fig. 16

Load factor – 1.75

Collapse loads

$v = 17.5$ tons

$H = 10.50$ tons

Ignoring the redundant horizontal reaction H_A, $H_D = H = 10.5\ T$.

The B. M. due to collapse restraining moments is of the form shown in Fig. 17.

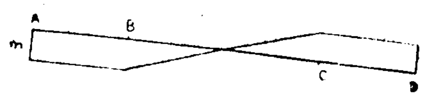


Fig. 17

The effect of reaction H_a is to produce a moment of the form shown in Fig. 18.

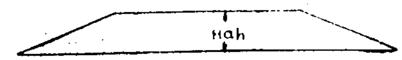


Fig. 18

Free collapse moment diagram is of the form, Fig. 19

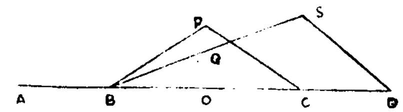


Fig. 19

$$OP = \frac{17.5 \times 30}{4}$$

$$= 131.25 \text{ ft-ton.}$$

$$CS = 10.5 \times 15$$

$$= 157.5 \text{ ft-ton.}$$

$$PQ = 131.25 - \frac{157.5}{2}$$

$$= 52.5 \text{ ft-ton.}$$

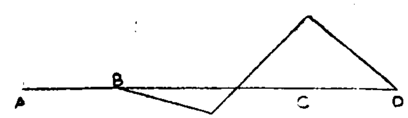


Fig. 20

Superimposition of the above diagrams will give the diagram shown in Fig. 21.

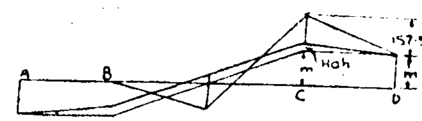


Fig. 21

$$m = Hah + 52.5 = 157.5 - m - Hah$$

$$m = \frac{157.5 - Hah}{2} = Hah + 52.5$$

$$\frac{3}{2} Hah = 78.75 - 52.5 = 26.25$$

$$Hah = \frac{26.25}{1.5} = 17.5$$

$$m = 52.5 + 17.5 = 70 \text{ ft ton} = 840 \text{ in.-ton.}$$

$$Z_p = \frac{840}{15.25} = 55 \text{ in.}^3$$

$$Z_e = \frac{55}{1.15} = 47.8 \text{ in.}^3$$

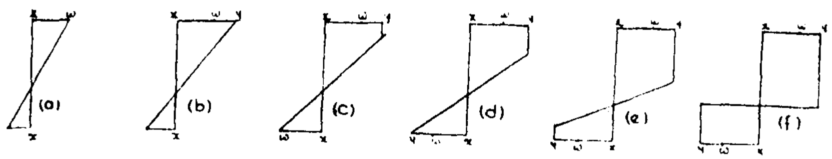


Fig. 22

14. Axial Load combined with Bending Moment

Fig. 22 shows the distribution of stress across the sections of members subjected to axial load combined with bending moment.

(a) Stress varies as its distance from the neutral axis. Extreme flange stress i.e. on compression side is less than the yield stress.

(b) Yield stress is reached on the compression side.

(c) Plasticity develops on the compression side. But tension zone is still in the elastic range.

(d) Yield stress is reached on tension side.

(e) Plasticity is developed on both sides.

(f) Section is fully plastic.

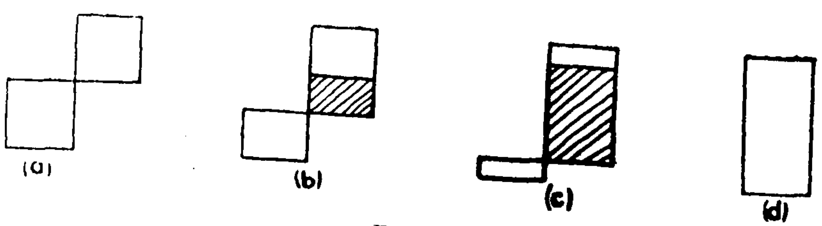


Fig. 23

Fig. 23 (a) gives the stress distribution across a fully plastic section under B. M. only.

Fig. (d) gives stress distribution due to direct load only.

Fig. (b) must give corresponding stress distribution for a fully plastic section under a large B. M. with a small direct load and Fig. (c) a small B. M. with a large direct load.

When the direct load is central, the rectangle representing tension on left hand side must be balanced by a rectangle representing compression on right hand side and that the shaded area represents the direct load.

The P. M. R. of the section gets reduced by an amount equal to P. M. R. of the portion under direct load.

If axial load $P = 25$ tons.

Collapse load $= 25 \times 1.75 = 43.75$ tons.

$$\text{Area of web required} = \frac{43.75}{15.25} = 2.87 \text{ in.}^2$$

$$\text{For } \tfrac{1}{2} \text{ in. thick plate, depth} = \frac{2.87}{0.5} = 5.74 \text{ in.}$$

P. M. R. of this portion of web

$$= \frac{\tfrac{1}{2} \times (5.74)^2}{4} \times 15.25 = 63 \text{ in.-ton.}$$

P. M. R. of an I-beam which carries a load of 43.75 tons at collapse is reduced by 63 in. tons.

If Z_p = plastic modulus of section,

available P. M. R. $= Z_p \times 15.25 - 63$.

15. Method of Virtual Work

A practical method of computing plastic collapse loads is suggested by Neal and Symmonds. This method consists of building up the actual collapse mechanism from a certain numbers of independent components which are termed as partial collapse mechanisms. A simple example will illustrate the method.

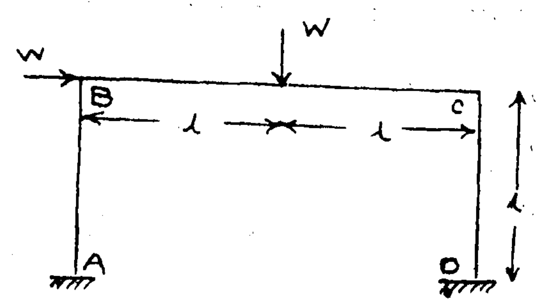


Fig. 24

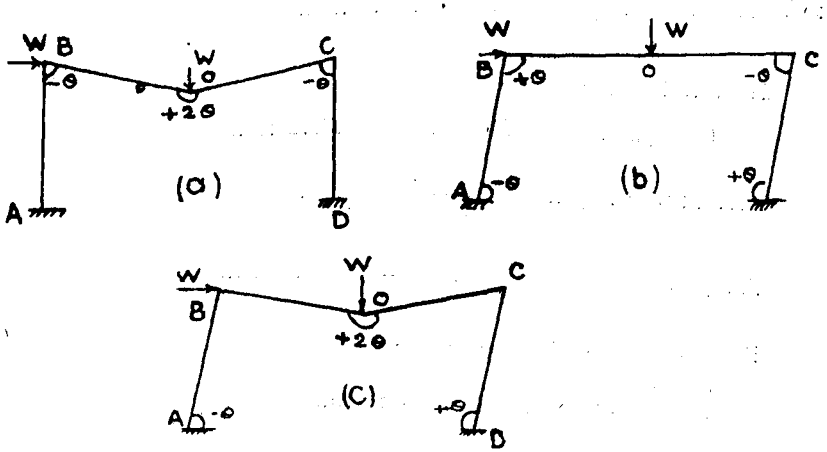


Fig. 25

In the above figures magnitudes of plastic hinge rotations are all shown in terms of a parameter θ.

A hinge rotation is positive if the hinge is opening when viewed from inside the frame.

In this case there are only three possible collapse mechanisms. For each mechanism it is possible to calculate the value of W by applying the principle of virtual work namely. 'Virtual work done by applied loads during a small displacement of the mechanism is equal to virtual work absorbed in the plastic hinges.'

It should be noted that work absorbed in any hinge is always positive.

Mp – plastic hinge moment.

In Fig. 25(a) work done by load $= W \times l\theta$

work absorbed $= Mp.4\theta$

$\therefore \quad W \times l\theta = Mp.4\theta \quad \therefore \; w = \dfrac{4Mp}{l}$

Fig. 25(b) $W \times l\theta = 4.\ Mp \times \theta \;\therefore w = \dfrac{4Mp}{l}$

Fig. 25(c) $W \times l\theta + W \times l\theta \quad = 6Mp \times \theta$

$$W = \frac{3Mp}{l}$$

The correct collapse mechanism can now be ascertained by applying 'Kinematic principle of plastic collapse' which states that for a given frame and loading the correct collapse mechanism is that mechanism to which collapses under the smallest possible value of the applied loads.

Least value of the applied loads $= \dfrac{3Mp}{l}$ and Fig 25(c) indicates the correct collapse mechanism.

Solution given above is checked by statics by making use of the Principle of uniqueness of solution which states that if a sufficient numbers of plastic hinges occur in a frame to transform the frame into a mechanism and if a B.M. diagram can be constructed for the frame in which the fully plastic moment occurs at each plastic hinge position, then the corresponding load is the correct collapse load if the fully plastic moment is not exceeded in the frame.

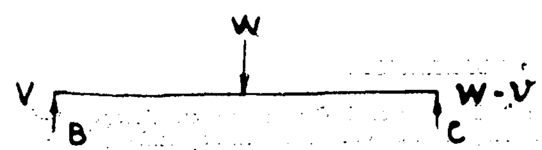


FIG. 26

$M_o - M_B = vl$

$M_o - M_C = (W-v)\,l$

Adding,

$2M_o - M_B - M_C = Wl \ldots \text{I}$

$M_B - M_A \quad = \dfrac{Wl}{2}$

$M_D - M_C \quad = \dfrac{Wl}{2}$

Adding,

$$M_B - M_A + M_D - M_C = Wl \qquad \ldots \text{II}$$

Fig. 26 $M_B = -M_P \qquad M_O = M_P \qquad M_C = -MP$

$$2M_P + M_P + M_P = 4M_P = wl$$

$$W = \frac{4M_P}{l}$$

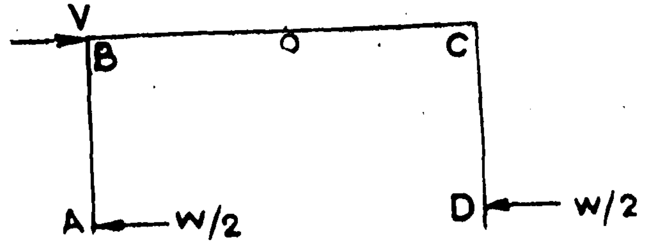


FIG. 27

Fig. 27 $M_A = -M_P, \quad M_B = M_P, \quad M_C = -M_P, \quad M_D = M_P$

$$4M_P = Wl \qquad W = \frac{4M_P}{l}$$

Adding I & II

$$2M_o - M_A - 2M_C + M_D = 2Wl$$

$M_o = 2Mp$

$M_o = -2M_P$

$M_d = M_P$

$M_A = -M_P$

$$6M_P = 2Wl$$

$$w = \frac{3M_P}{l}$$

16. No reference has been made in this note to the influence of factors like shear, buckling, and rolling loads on the method of plastic design.

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NAME	YEAR	METHOD OF TESTING	STRESS-STRAIN CURVE	DISTRIBUTION OF STRESS: ELASTIC	PARTIALLY PLASTIC	FULLY PLASTIC	REMARKS
WICKSEED	1866	An autographic recording machine was used for measurement of extension of tension test piece and load was measured by hydraulic pressure.					Some resemblance of drop in stress at yield is observed. Error was introduced on load side due to friction of the ram.
EWING	1899	Hydraulically operated testing machine was used.			ft		A slight drop in stress at yield was obtained, but he did not give any significance to it.
ROBERTSON & COOK	1913	Specimen was loaded through special shackles having a ball and anvil device to ensure axiality and set between two parallel steel rods of greater strength with which it shared the load.		fu	ft fu	ft	True stress strain diagram was obtained.
KENNEDY	1923	Two specimens, one of rectangular cross-section and other of square cross-section were tested.					It was shown that the ratio of stress for load at collapse to tensile yield stress depends upon the geometry of the cross-section.
MUIR & BINNIE	1926						No drop in stress at yield was observed. This was due to care not being taken to ensure axial loading.
HAIGH	1934		LOWER YIELD POINT	ft	ft	ft	It was considered that the lower yield stress was significant and that this depended upon the rate of straining.
RINAGL	1936	Tests were performed both in tension and in flexure.		ft	ft fu	ft	Upper yield stress obtained in flexure was 20 per cent higher than that obtained in tension. This was because of small unavoidable eccentricities in the latter test.
BAKER & RODERICK	1938	Tests were performed on simply supported beams with a central concentrated load.		ft	ft	ft	Lower yield stress calculated from collapse loads was always lower than that obtained in corresponding tension test.
MORRISON	1939	Stresses at yield in tension, compression, and flexure were determined.		fu	ft fu	ft	Yield phenomenon was identical in tension and in compression.

1 USE OF LIME IN SOIL STABILIZATION

2 LABORATORY EXPERIMENTS ON USE OF PRIMERS IN PREVENTING CAPILLARY RISE

USE OF LIME IN SOIL STABILIZATION

By

S. R. MEHRA & L. R. CHADDA

CONTENTS

SYNOPSIS

Results of laboratory experiments on the effect of the addition of different percentages of lime to four typical soils of the Punjab on the physical characteristics such as optimum moisture content, resistance to the softening action of water, shrinkage, and compressive strength are reported. Penetration resistance of lime-soil blocks when cured for different periods are given. It is concluded that the addition of lime improves compressive strength, reduces shrinkage, and increase resistance to the softening action of water. Sandy loams react better to the lime treatment than silty or silty clay loams.

1. INTRODUCTION

1.1. Cement has been employed in stabilization of soil with success. Due to high cost of cement, its application on a large scale is limited except in emergency construction. Lime is one of the oldest construction materials and is extensively employed in building construction. So laboratory experiments have been undertaken to explore the possibility of use of lime in soil stabilization. The effect of different quantities of lime on the compressive strength, resistance to the softening action of water and other physical characteristics of various lime-soil mixtures have been studied. Results obtained are presented in this note.

In all the experiments hydrated lime (commercial) has been used. Physical characteristics of the soils selected for the study are given in Table 1.

TABLE 1

Physical Characteristics of Soils

S. No.	Type of soil	Liquid Limit	Plasti-city Index	Sand content	Optimum moisture	Total solids
1.	Sandy loam	21.6	6.3	64.5	12.0	0.083
2.	Silty loam	26.9	9.4	13.9	11.0	0.133
3.	Silty clay loam	36.7	16.2	11.5	11.0	0.242
4.	Loam	29.7	10.7	43.1	11.0	0.114

2. OPTIMUM MOISTURE OF LIME-SOIL MIXTURES

2.1. For successful stabilization it is essential that work should be conducted at optimum moisture. So the effect of increasing percentages of lime on the optimum moisture of all the four soils was studied and results are given in Table 2.

TABLE 2

Optimum Moisture of Lime-soil Mixtures

Serial No.	Type of soil	Optimum Moisture for Percentage of lime				
		0	2.5	5.0	7.5	10
1.	Sandy loam	12.0	1.40	14.0	14.0	15.0
2.	Silty loam	11.9	12.0	12.0	12.0	13.0
3.	Silty clay loam	11.0	13.0	13.0	13.0	13.0
4.	Loam	11.0	12.0	12.0	13.0	13.0

It will appear from Table 2 that there is slight increase in the optimum moisture with the increased addition of lime.

3. RESISTENCE OF LIME-SOIL MIXTURES TO THE SOFTENING ACTION OF WATER

3.1. Specimens were made with all the four soils by compacting 200 grams of dry soils at optimum moisture. The blocks were made in compaction apparatus with 5½ lb hammer dropping through a height of 15 in. with 40 strokes. The compacted lime-soil blocks were cured in humid air for a fortnight and subsequently dried in an oven at 50°-60° C to constant weight.

3.2. The blocks were immersed in water at 25° C and the percentage loss in weight through disintegration after 5 hours immersion and drying was recorded. This is given in Table 3.

TABLE 3.

Disintegration of Lime-Soil Blocks under Water

Per-centage lime	Percentage of soil disintegrated in 5 hours			
	Sandy loam	Silty loam	Silty clay loam	Loam
0	Failed in 15 minutes	Failed in 15 minutes	Failed in 40 minutes	Failed in 30 minutes
2.5	0.51	3.53	4.82	3.09
5.0	Nil	0.51	1.2	0.50
7.5	Nil	0.50	1.0	0.50
10.0	Nil	0.50	1.5	Nil

It will appear from Table 3 that addition of 2.5 percentage lime considerably improves the water resistance quality of all the four soils. Sandy loam shows least disintegration while silty clay loam the maximum. With the addition of higher percentage of lime, blocks of all the four soils showed increased resistance to the detrimental action of water.

4. EFFECT OF LIME ON THE PLASTICITY OF SOILS

4.1. Experiments were performed to study the effect of increasing quantities of lime on the plasticity index of all the four soils. Results obtained are given in Table 4 and shown in Fig. 1

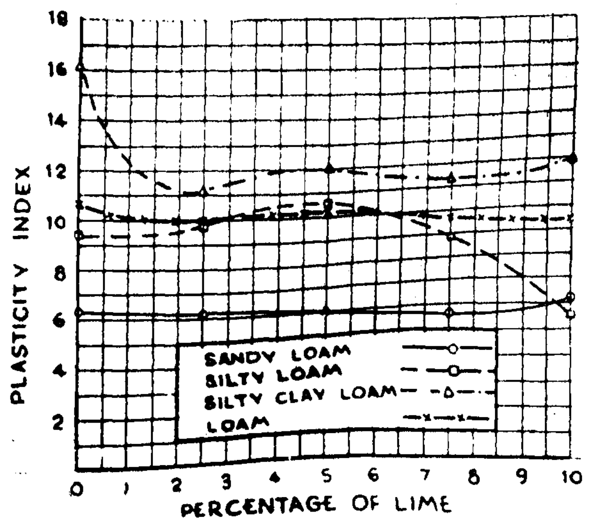

Fig. 1. Effect of Lime on the Plasticity Index

TABLE 4

Effect of Lime on the Plasticity Index of Soils

S. No.	Type of Soil	P.I. with percentage of lime				
		0	2.5	5.0	7.5	10.0
1.	Sandy loam	6.3	6.2	6.0	5.6	6.2
2.	Silty loam	9.4	9.8	10.6	8.9	5.6
3.	Silty clay loam	16.2	11.2	12.1	11.3	11.9
4.	Loam	10.7	10.0	10.3	9.7	9.4

4.2. It will appear from Table 4 that only silty clay loam has a tendency to show marked reduction in plasticity index with addition of lime, while the others do not show any marked change. This would appear to indicate that the pulverisation of fat clays may be rendered much easier when mixed with lime.

5. SHRINKAGE OF COMPACTED LIME-SOIL MIXTURES

5.1. Shrinkage of a soil increases as the moisture of compaction increases and varies with the nature of the soil. When a soil is to be used in engineering construction, it is very essential that its shrinkage on drying should be as little as possible.

5.2. All the four soils mentioned in Table 1 were compacted with increasing percentage of lime at the corresponding optimum moisture. The volume at the time of compaction and after curing in humid air for a fortnight with subsequent drying in the oven at 50° C were determined. Results obtained are given in Table 5.

TABLE 5

Effect of Lime on the Shrinkage of Soils

S. No.	Soil	Percentage Lime	Percentage volumetric shrinkage	Density
1.	Sandy loam	0	1.18	1.96
		2.5	1.03	1.88
		5.0	0.93	1.86
		7.5	0.83	1.84
		10.0	0.72	1.81
2.	Silty loam	0	1.6	1.92
		2.5	1.0	1.92
		5.0	0.80	1.79
		7.5	0.64	1.78
		10.0	0.52	1.75
3.	Silty clay loam	0	2.8	1.94
		2.5	2.2	1.88
		5.0	2.0	1.84
		7.5	1.38	1.81
		10.0	1.23	1.79
4.	Loam	0	1.56	1.95
		2.5	0.78	1.85
		5.0	0.63	1.83
		7.0	0.63	1.82
		10.5	0.62	1.79

It will appear from Table 5 that the shrinkage of compacted oil decreases as the percentage of lime increases.

5.3. The effect on shrinkage by the addition of small ercentages of Plaster of Paris (deydrated gypsum) Plus varying ercentage of lime has also been tried. Results obtained are given ı Table 6.

TABLE 6

Effect of Lime plus Plaster of Paris on the Shrinkage of Soils

TABLE 6 (*Contd.*)

S. No.	Soil No.	Percentage Lime	Percentage Plaster of Paris	Density	Percentage shrinkage or expansion
1.	Sandy loam	0	0	1.96	−1.18
		2.5	0.5	1.86	Nil
		,,	1.5	1 87	Nil
		,,	2 5	1.87	−.09
		,,	5.0	1.87	Nil
		5.0	0.5	1.84	−.18
		,,	1.5	1.84	−.09
		,,	2.5	1.84	− 09
		,,	5.0	1.81	+.177
		7.5	0.5	1.83	− 089
		,,	1.5	1.81	−.88
		,,	2.5	1.84	Nil
		,,	5.0	1.82	+0.269
		10.0	0.5	1 81	Nil
		,,	1.5	1 80	+.044
		,,	2.5	1.82	+.178
		,,	5.0	1.78	+.218
2.	Silty loam	0	0	1.92	−1.92
		2.5	0.5	1.79	−.44
		,,	1.5	1.76	+2.09
		,,	2.5	1.78	+2.33
		,,	5.0	1 75	+3.05
		5.0	0.5	1.76	+.218
		,,	1.5	1 71	+3.33
		,,	2.5	1.72	+3.91
		,,	5.0	1.68	+4.60
		7.5	0.5	1.73	+ .47
		,,	1.5	1.67	+3.6
		,,	2.5	1.65	+4.13
		,,	5.0	1.67	+3.74
		10.0	0.5	1.75	+ .53
		,,	1.5	1.70	+3.10
		,,	2.5	1.69	+3.76
		,,	5,0	1.68	+2.88
3.	Silty clay loam	0	0	1.94	−2 8
		2.5	0.5	1 85	−1.33
		,,	1.5	1.77	+ .22
		,,	2 5	1.79	+ .31
		,,	5.0	1.72	+1.38
		5.0	0.5	1.76	−1.24
		,,	1.5	1.75	+ .94
		,,	2.5	1.72	+1.26
		,,	5.0	1.67	−1.97
		7.5	0.5	1.76	− .86
		,,	1.5	1.73	+ .75
		,,	2.5	1.73	+3 1
		,,	5.0	1.72	+3.6
		10.0	0.5	1.73	− .67
		,,	1.5	1.72	+ .97
		,,	2.5	1.68	+3.92
		,,	5.0	1.67	+3.29
4.	Loam	0	0	1.95	−1.56
		2.5	0.5	1.84	+ .046
		,,	1.5	1.83	+ .18
		,,	2.5	1.83	+1.26
		,,	5.0	1.84	+1.07
		5.0	0.5	1.80	− .089
		,,	1.5	1.79	+ .48
		,,	2.5	1.78	+ .70
		,,	5.0	1.80	+1.24
		7.5	0.5	1.79	+.53
		,,	1.5	1.77	+.44
		,,	2.5	1.82	+.36
		,,	5.0	1.82	+.72
		10.0	0.5	1.76	+.008
		,,	1.5	1.75	+.17
		,,	2.5	1.74	+.085
		,,	5.0	1.73	+.17

\+ indicates expansion. − indicates shrinkage.

5.4. It will appear from Table 6 that an addition of Plaster of Paris as low as 0.5 per cent and 2.5 per cent or more of lime not only eliminates shrinkage but also produced expansion in some soils. With higher percentages of Plaster of Paris there is expansion in almost all soils. Fine grained soils (No. 2 & 3) show comparatively greater expansion than coarse grained ones (Nos. 1 & 4).

Effect of Lime plus Plaster of Paris on the Water resistance of soils was also studied. The soil blocks prepared for shrinkage test were subjected to disintegration test. The blocks were immersed in water for 5 hours and their disintegration studied. Results obtained are given in Table 7.

TABLE 7

Disintegration of Soil Blocks mixed with Lime plus Plaster of Paris

Soil No.	Percentage lime	Percentage plaster of paris	Density	Percentage loss after 5 hours immersion in water	Remarks
1.	2.5	.5	1.86	0.5 increase	Too many cracks
	,,	1.0	1.86	0.75 loss	Remained intact
	5.0	.5	1.84	1.34 increase	Cracks
	,,	1.0	1.83	No loss.	Remained intact
2.	2.5	.5	1.79	7.5 loss	Disintegrated
	,,	1.0	1.75	2.0 loss	Slight cracks
	5.0	.5	1.76	2.27 increase	Intact
	,,	1.0	1.76	No loss	Cracks without disintegration
3.	2.5	.5	1.85	0.037 increase	-do-
	,,	1.0	1.81	0.62 increase	Badly cracked
	5.0	.5	1.76	3.05 increase	Intact
	,,	1.0	1.72	1.37 increase	Cracks
4.	2.5	.5	1.84	25.5 loss	Disintegration
	,,	1.0	1.77	1.95 increase	Intact
	5.0	.5	1.80	1.77 increase	Cracks without disintegration
	,,	1.0	1.78	0.50 loss	Intact

It will appear from Table 7 that the presence of small percentages of Plaster of Paris adversely affects the water-resisting quality of lime-soil compacted blocks, and as such its addition in combination with lime cannot be ordinarily-recommended.

5.5. Compressive Strength of Lime-Soil Mixtures

The effect of varying concentrations of lime on the compressive strength of different soils was studied. All the four types of soil mentioned in Table I were compacted with 2.5, 5.0, 7.5 and 10 per cent lime at the corresponding optimum moisture in the compaction apparatus*. The blocks were cured in humid air for 15 days, dried in the sun, and tested for compressive strength. Also experiments were conducted to find out the effect of prolonged curing on the compressive strength of lime-soil blocks. Lime-soil blocks prepared at optimum moisture were cured for 15 days in humid air and then kept over wet sand in contact with capillary moisture for one month and subsequently dried to constant weight in the oven at 50° C. The results obtained are given in Table 8.

TABLE 8

Compressive Strength of Lime-Soil Mixture

Soil	Percentage lime.	Compressive Strength of blocks cured for 15 days in lb per Sq. in. at		Compressive Strength of blocks after humid curing for 15 days and kept over wet sand for one month	
		Initial crack	Final crack	Initial crack in lb per sq. in.	Final crack in lb per sq. in.
Sandy loam	0	149.3	300 2	149 3	300.2
	2.5	149.3	237.4	387.5	562.6
	5.0	179.2	327.0	443.5	664.0
	7.5	268.8	403.2	537.6	810.8
	10.0	292.0	421.1	597.1	850.8
Silty loam	0	268.8	520.7	268.8	520.7
	2.5	268.8	501.7	161.2	161.2
	5.0	292.0	528.6	104.0	387.0
	7 5	358.0	743.7	597.0	806.0
	10.0	380.8	788.4	582.0	1102.0
Silty clay loam	0	538.0	694.0	538.0	694.0
	2.5	448.0	627.0	336.0	414.8
	5.0	538.0	761.6	177.0	416.0
	7.5	627.0	985.6	174.3	506.0
	10.0	851.2	1048.0	137.0	552.0
Loam	0	470.0	775.0	470.0	775.0
	2 5	220.0	506.0	343.0	528.6
	5 0	286.0	530.0	364.2	579.2
	7.5	350.0	540.0	432.0	752.6
	10.0	366.0	556.0	244.0	558.0

* For details see Reference 1 on page 494.

5.6. It will appear from Table 8 that the addition of lime produces some increase in the compressive strength of soils. Curing for a longer period produces increase in the strength of lime-soil blocks prepared from sandy loam. With higher percentages of lime, silty loam also gives fairly high strength.

6. EFFECT OF CURING ON PENETRATION RESISTANCE

6.1. Lime-soil blocks prepared from different soils and mixed with increasing percentages of lime were cured under varying conditions and periods of time and their penetration resistance in saturated state with Proctor's needle and absorption studied.

6.2. In the first set of experiments, the lime-soil blocks prepared at optimum moisture with increasing percentages of lime were kept in humid air for a fortnight. Afterwards these blocks were placed in perforated iron moulds and kept over wet sand for 48 hours.

In another set of experiments lime-soil blocks were cured for a fortnight in humid air and kept over wet sand for a period of one month.

In the third set of experiments, the blocks were cured in humid air for a period of one month and kept over wet sand for 48 hours. The results are given in Table 9.

It will appear from Table 9 that with increasing percentage of lime there is an appreciable increase in the penetration resistance of all the four sorts under wet conditions. Also, extending the period of humid curing and/or contact with capillary moisture results in substantial increase in strength.

TABLE 9

Absorption and Penetration Resistance of Lime-Soil Blocks cured by Different Methods

Soil No.	Lime percent	Absorption Percent			Penetration Resistance Lb per sq. in.		
		Moist curing for 15 days and kept over wet sand for 48 hours	Moist curing for 15 days and kept over wet sand for one month	Moist cured for one month & kept over wet sand for 48 hours	Moist curing for 15 days and kept over wet sand for 48 hours	Moist curing for 15 days and kept over wet sand for one month	Moist cured for one month & kept over wet sand for 48 hours
Sandy loam	0	15.9	16.7	15.8	72	146	120
	2 5	14.0	14.2	12.5	380	720	540
	5.0	13 6	15.5	13.1	640	800	840
	7.5	11.7	16.2	11.8	960	1260	940
	10.0	14.7	15.7	13.7	960	1340	1640
Silty loam	0	24.6	22.9	26.0	24	24	—
	2.5	16.0	17.5	15.2	1380	1600	1120
	5.0	17.0	18.8	17.5	1520	above 2000	above 2000
	7.5	17.6	19.6	15.2	above 2000	,,	,,
	10.0	19.2	19.9	17.6	,,	,,	,,
Silty clay loam	0	20.2	22.7	24.9	32	36	54
	2.5	18.2	20.6	15.8	720	1600	1360
	5.0	19.0	20.1	18.6	1320	1800	1940
	7.5	21.1	22.3	24.9	1480	above 2000	above 2000
	10.0	22.0	21.1	20.4	1520	,,	,,
Loam	0	20.1	16.5	18.8	20	124	92
	2 5	13.6	16.2	13.7	540	800	1160
	5.0	15.9	16.9	14.6	940	1840	1600
	7.5	17.1	15.6	16.6	980	above 2000	2000
	10.0	17.6	18.3	16.6	1180	,,	2000

7. CONCLUSIONS

The above study conducted with lime-soil mixture has shown that small quantities of lime when added to a sandy soil considerably increases its resistance to the softening action of water and compressive strength. Shrinkage of soils is also appreciably reduced by the addition of lime thereby increasing its utility as a construction material. The addition of lime to plastic soils reduces their plasticity index making them friable; consequently handling during construction is made easier. Addition of small quantities of lime in the stabilized soil base courses may be highly beneficial in preventing softening due to capillary moisture in areas of high sub-soil water level.

SUMMARY

Effect of lime on the physical characteristics of different soils has been investigated, it has been shown that the addition of small quantities of lime to a soil improves its strength as well as its water-resistance quality. Sandy loam soils react better than silty or silty clay loam soils.

ACKNOWLEDGMENT

Shri B. P. Kapur and Shri O. P. Wason assisted the Authors in the above investigations.

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LABORATORY EXPERIMENTS ON USE OF PRIMERS IN PREVENTING CAPILLARY RISE

By

S. R. MEHRA, H. L. UPPAL & L. R. CHADDA

In areas of high water table (called water-logged areas) due to capillary rise, the soil gets wet up to a level close to the surface. Due to reduction in bearing power a wet subgrade is not capable of taking much load, with the result that generally roads constructed in such areas either fail or are seriously deformed. When the road structure is designed to be adequate for the reduced bearing power when the soil gets softened due to capillarity it has to be very thick, and therefore becomes very expensive. It is, therefore, desirable to find some method of checking the capillarity of the soil at a desired depth so as to keep the soil comparatively dry and capable of taking load.

2. Experiments have, therefore, been carried out to find out the effectiveness of the application of cold penetration bituminous materials on the surface of the subgrade in checking capillarity. The soil selected for the purpose is a silty clay loam, liquid limit 31.0, plasticity index 12.6 and sand content 17.5, is typical of the silty soils generally met with in the Punjab. The soil is compacted to a thickness of 3 inches in a metallic box with a hole at the bottom. Dry bulk density of 1.6 which is the lower limit of density specified for subgrades is adopted.

3. The subgrade specimens were allowed to dry and then painted with three grades of cold penetration bituminous material available in the market at the rate of 20, 25, 35 and 40 lb per 100 sq. ft. The bituminous material was warmed slightly before application. After it had soaked in, which takes 2-3 days, another 3 in. layer of soil was compacted over it at optimum moisture to attain a density of 1.8 which is the generally specified density for base courses. The analysis of the soil used for the top layer was, liquid limit 27, plasticity index 10.4, and sand content 47.5. A soil with higher plasticity index than generally used for stabilized soil base course was purposely used to quicken absorption of moisture in case of failure of the painted surface. A number of such specimens were made and the bottom holes were connected to a common reservoir, the top level of which was maintained at half an inch below the painted surface,

Fig. 1. The experiment was watched from day to day and the time when the base course got wet to the top on account of

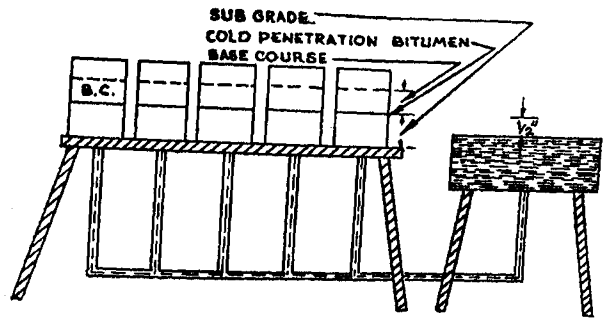

Fig. 1.

the failure of the painted surface was recorded. The results are given in Table 1.

TABLE 1

Time Painted Surface to Fail Under Capillary Action

Cold penetration bituminous material	Rate of painting lb per 100 sq. ft.	Time of failure
(Primer No. 1)	20	Not affected for over 11 months
,,	25	do.
,,	35	do.
,,	40	do.
(Primer No. 2)	20	2 months
,,	25	2 months 2 days
,,	35	3 months 20 days
,,	40	Undisturbed for 6 months
(Primer No. 3)	20	Three days
,,	25	Six days
,,	35	3 months 20 days
,,	40	Undisturbed for 9 months

4. Composition of Primers used.

Primer No. 1

Bitumen 80/100	50 per cent
Diesel Oil	50 per cent

Shell Primer No. 2

Bitumen 80/100	30 per cent
Diesel Oil	70 per cent

Shell Primer No. 3

Bitumen 80/100	30 per cent
Furnace Oil	70 per cent

It will appear from Table 1 that the application of Primer No. 1 at 20 lb per 100 sq. ft. can check capillarity for a long time, but the same rate of application of Primer No. 2 & 3 failed to create that much resistance. Higher rates of the application of latter two were tried and were found to be successful, but were rather uneconomical.

SUBGRADE IMPREGNATED WITH SODIUM SULPHATE

5. It has been observed that in areas of high water table the subgrade is frequently impregnated with sodium sulphate, a salt which is well known for its detrimental action on soil. Presence of more than 0.10 per cent of the salt is considered harmful. The tests reported in Paras. 2 & 3 for checking capillarity have, therefore, been repeated by impregnating the subgrades with 0.20 percent sodium sulphate. The subgrade was painted with all the three bituminous primers at 20, 25, 35 and 40 lb per 100 sq. ft. The time taken in each case for the capillary force to puncture the painted surface is given in Table 2.

TABLE 2

Time taken by the Painted Surface to fail when the Subgrade contains Sodium Sulphate

Cold penetration Bituminous material	Rate of painting lb per 100 sq. ft.	Time
(Primer No. 1)	20	1 month and 28 days
,,	25	4 months and 20 days
,,	35	Undisturbed for 5½ months
,,	40	do.
(Primer No. 2)	20	12 days
,,	25	24 days
,,	35	28 days
,,	40	3 months and 3 days
(Primer No. 3)	20	4 days
,,	25	7 days
,,	35	9 days
,,	40	One month and 22 days

6. It will appear from Table 2 that Primer No. 2 and 3 when applied at 40 lb per 100 sq. ft. or less cannot check capillarity when sodium sulphate is present in the subgrade, whereas primer No. 1 if applied at 35 lb per 100 sq. ft. is effective. It will, therefore, appear that testing for the presence of the salt is very essential before a cold penetration bituminous material is recommended for use in subgrade to check capillarity.

7. Tar No. 3 applied at the rate of 35 lb per 100 sq. ft. was also tried. There appeared to be no sign of the base course getting wet for months.

8. Field experiments to find out the effect of traffic on the stability of the painted subgrades are indicated before definite conclusions are drawn.

Paper No. 179

THE INFLUENCE OF A SUPERIMPOSED LAYER IN INCREASING THE LOAD BEARING CAPACITY OF A RIGID PAVEMENT

By

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M. R. SAN. I. (Lond.)

CONTENTS

SYNOPSIS

The effect of a superimposed layer consisting of a rigid or a flexible construction or a combination of both for strengthening rigid pavements has been discussed. Stresses due to temperature changes have been detailed. The general conclusion is that a flexible superimposed layer increases the load bearing capacity of an existing rigid pavement greater than a rigid pavement. Examples illustrating the strengthening of airfield pavements are worked out.

The advent of jet aircraft has made the choice of suitable pavement materials difficult. The methods adopted to enable bituminous materials to withstand the effects of jet blast and fuel spillage have been discussed. Recently published test reports from the U. S. A. and the U. K. which are in general agreement with the theoretical analysis, are summarised.

The need for further investigations on temperature stress under conditions obtaining in India is stressed and suggestions for the addition of certain indigenous materials as filler for resisting the blast effects of jet planes is offered.

NOTATION

Symbol	*Description*
A_i, A_e & A_w	– Stress amplitude in temperature stresses suffixes i, e and w denote interior, edge and corner loading cases respectively.
a	– $\sqrt{\left(\frac{P}{\pi p}\right)}$, radius of the surface of contact between wheel and slab.
B	– Free length or width of a slab when subjected to warping.
C	– $\frac{E_s}{1-\mu_s^2}$ Modified modulus of elasticity of subgrade.
D	– $\frac{E_p\ h^3}{12\,(1-\mu_p^2)}$ Stiffness of pavement.
E	– Modulus of elasticity.
E_p	– Modulus of Elasticity of pavement.
E_s	– Modulus of elasticity of Subgrade.
e	– The coefficient of linear expansion of concrete.
h	– Thickness of pavement.
k	– $\frac{p}{w}$ Modulus of subgrade reaction.
K_e	– Coefficient of end restraint. Unity for total restraint.
K_w	– Coefficient of warping restraint.* K_{w1} and K_{w2} are coefficients relating to breadth and length of the slab respectively.

* This may rise to 1.1 according to Westergaard for complete resistance to warping).

l	– $\sqrt[4]{\frac{D}{K}}=\sqrt[4]{\left(\frac{E_p\ h^3}{12\,(1-\mu^2_p)\ k}\right)}$
p	– Uniformly distributed load over the loaded area, i.e. contact pressure between wheel and slab.
P	– Wheel Load
r	– Distance from the point of application of a load to the point at which stresses are computed.
Sc	– Stress induced in a concrete slab due to corner loading in pounds per square inch.
S_i	– Stress induced by load in a concrete slab due to interior loading in pounds per square inch.
S_t	– Maximum stress due to temperature differential in pounds per square inch.
t	– Temperature differential in degrees Fahrenheit.
t_s, t_i & t_w	– Phase time for restrain temperature stress, for internal temperature stress due to curved temperature gradient and for warping restraint temperature stress respectively.
θ_0	– Amplitude of the assumed temperature variation curve of the surface$=\frac{1}{2}$ the total variation when the curve is assumed to be sinusoidal.
θ'_0	– Instantaneous value of at a specified time.
θ'_d	– The temperature difference between the top and bottom faces of the slab at its maximum value.
μ	– Poisson's ratio.
μ_p	– Poisson's ratio for pavement.
μ_s	– Poisson's ratio of subgrade.
w	– Vertical deformation.

1. INTRODUCTION

1.1. The Problem

Various theories on the stress-strain conditions in soils and pavement materials have been lately advanced for designing concrete pavements. While these theories are useful in making a rational design of new pavements, the problem is complicated when an existing concrete pavement has to be strengthened to support heavier wheel loads and higher tyre pressure. Since it is not possible to improve the subgrade or the sub-base lying underneath an existing pavement, the only course open is to strengthen the pavement by a superimposed layer.

1.2. The Scope of the Paper

Consideration of the problem of strengthening an existing concrete pavement by a superimposed layer (overlay) has been restricted to the two principal factors, viz.,

(a) the effect of wheel load on pavement, and
(b) stresses due to temperature variation.

The methods discussed are superimposition of :

(a) a concrete overlay,
(b) a flexible overlay, or
(c) a combination of the two, i.e., a concrete overlay intercoursed with a thin layer of a bituminous carpet.

1.3. For convenience, the Paper has been divided into four parts, namely, theoretical analysis, the application of the analysis to strengthening of airfield pavements subject to jet blast and fuel spillage, review of some recent tests, and general conclusions.

THEORETICAL ANALYSIS

2. GENERAL REVIEW OF THEORIES FOR CALCULATION OF STRESSES

2.1. Elastic and Liquid Theories

In all the theories the characteristics of soil and the structural action of the sub-grade have been idealised even though the soil characteristics are subject to wide variations and the subgrade structure is far from homogeneous.

There are two fundamentally different theories for the calculation of stresses and deflections of concrete pavements, viz., the elastic theory of soil and the liquid theory of soil.

2.2. The Elastic Theory

According to this theory, the soil is regarded as an elastic, isotropic, homogeneous and semi-infinite body. It issumed that the deformation of the subgrade is not uniform under a load and that it extends beyond the loaded area. On the basis of this theory, the pressure distribution at the edge of a loaded area is large, and would cause plastic deformation at the edge even for small loads.

The characteristics of soils which influence the stresses in concrete pavement according to this theory are :

(i) Modulus of elasticity of the subgrade.
(ii) Poisson's ratio of the subgrade.

2.3. The Liquid Theory

The sub-grade is regarded as an elastic foundation and the relation between the pressure p and deformation w at all points is assumed to be governed by the relation $p = k.w$, where k is the modulus of soil reaction (Westergaard's theory). The deformation being uniform, the pressure distribution is also assumed to be uniform.

The fundamental difference between the two theories is diagrammatically shown in Figs. 1 and 2.

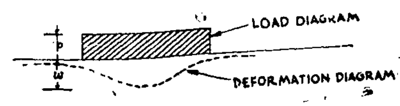


Fig. 1
Load and deformation diagrams according to the elastic theory.

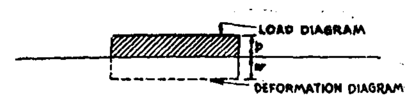


Fig. 2
Load and deformation diagrams according to liquid theory.

3. THE ELASTIC THEORY OF SOIL

3.1. The assumptions made and the limitations of the elastic theory together with the method of calculation of stresses in a two layered system advanced by Boussenesq and Burmister are given in Appendix 1.

3.2. It is seen from Appendix 1 that the load bearing capacity of a pavement is dependent not so much on the absolute value of the modulus of elasticity of pavement as on the ratio of the modulus of elasticity of pavement to the modulus of elasticity of the subgrade. In strengthening of an existing pavement it might, therefore, be assumed, to a first approximation, that a superimposed layer which has a modulus of elasticity much higher than the modulus of elasticity of the existing pavement is more effective than a superimposed layer having the same modulus of elasticity as the existing pavement.

The value of 'a' is fixed by the contact area of the wheel. The value of 'h' is restricted by considerations of cost. The engineer has, therefore, to formulate his design (within the practical range of variations in the ratios $\frac{E_p}{E_s}$ and $\frac{a}{h}$. If the ratio $\frac{E_p}{E_s}$ is low, say 5, the strength property of the subgrade has a large influence on the supporting capacity of the system. On the other hand, if the ratio $\frac{E_p}{E_s}$ is high, say 1000, which is generally the case with a concrete pavement, the strength property of the pavement has a large influence on the supporting capacity as will be evident from Table 1.

TABLE 1

The influence of the ratio E_p/E_s on the load bearing capacity of a two layer system.

Thickness of pavement, h, in.	Max. bearing pressure in lb per sq. in. if $Es = 3000$, $E_p/E_s = 5$, $w = 0.2$ and $a = 15$ in.	Max. bearing pressure in lb per sq. in. if $E_s = 3000$, $E_p/E_s = 1000$, $w = 0.05$ and $a = 15$ in.
6	20	31.4
12	26	60.5
18	31	94.5
24	37.8	121.0

It is seen from Table 1 that the load bearing capacity increases nearly in the same proportion as the thickness of the pavement when the ratio E_p/E_s is high but the corresponding increases are slight when the ratio E_p/E_s is low.

The particulars in the last column in Table 1 represent those for a typical airfield pavement except that the base course has been omitted in order to apply Burmister's theory of two-layer system.

3.3. Computation of Stress Distribution in a three-layered system (Fox, Acum)

The effect of introducing a base course between the pavement and subgrade in reducing the vertical stress on the subgrade due to the applied load is clearly brought out by the computations made by Acum and Fox[6]. This is illustrated by the following examples.

Example 1:

Assume a two-layer system in which

$E_s = 3,000$ lb per sq. in.

$E_p = 3,000,000$ lb per sq. in.

$a = 15$ in.

$a/h = 2$ approximately

$h = 8$ in.

From Table 2 Road Research Technical Paper No. 9 by Fox[5] assuming perfectly rough interface, the vertical stress on the subgrade is $0.071\ p$, where p is the applied load on the pavement in lb per sq. in.

Example 2:

Assume a three-layer system in which the pavement, Fig. 3, has $h_1 = 8$ in.

$E_1 = 30,00,000$,

the base course has $h_2 = 16$ in., $E_2 = 30,000$ and the subgrade has $E_3 = 3,000$.

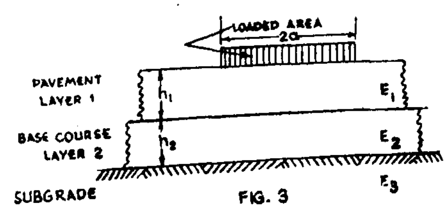


FIG. 3

From the notations used in the computation of stresses in a three-layer elastic system of Acum and Fox[6],

$$a_1 = \frac{a}{h_2} = 1$$

$$K_1 = \frac{E_1}{E_2} = 100$$

$$H = \frac{h_1}{h_2} = \tfrac{1}{2}$$

$$K_2 = \frac{E_2}{E_3} = 10$$

But $\dfrac{E_1}{E_3} = 1{,}000$ (as in Example 1)

Table 3 of Road Research Technical Paper No. 9 by Fox in which $H = \frac{1}{2}$ and $a_1 = 1$ is, therefore, applicable to the present case. From this table, for the above noted values of K_1 and K_2, the vertical stress at the first interface between the pavement and base course is $0.142\ p$ and at the second interface between the base course and subgrade is $0.047\ p$.

3.4. It will, therefore, be seen that by introducing a base course of thickness twice that of the pavement and having a modulus of elasticity 10 times that of the subgrade, the vertical stress on the subgrade is reduced from $0.071\ p$ to $0.047\ p$ or a reduction of 2.4 per cent of the applied load p.

Computations made by Acum and Fox are restricted to the following ranges :

$$K_1 = \frac{E_1}{E_2} \text{ from 5 to 500}$$

$$K_2 = \frac{E_2}{E_3} \text{ from 5 to 100}$$

$$a_1 = \frac{a}{h_2} \text{ from } \tfrac{1}{2} \text{ to } 1$$

$$H = \frac{h_1}{h_2} \text{ from } \tfrac{1}{4} \text{ to } 1.$$

Stresses at the first interface where $K_1 = 1$, i.e. in the case of superimposed layer having the same modulus as that of the existing pavement, requires the knowledge of the stress at points in the upper layer of a two layer system for which no computations have been performed by Acum and Fox.

3.5. The theory of layered system assumes that each layer is continuously in contact with the layer on which it is resting. In some cases a superimposed concrete layer is laid after a thin bituminous layer is spread over the existing pavement in order to prepare a smooth surface on which an even thickness of superimposed layer can be laid, Fig. 4.

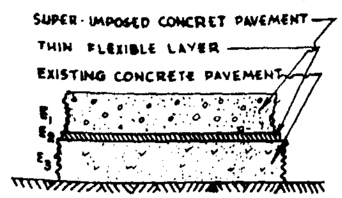


FIG. 4

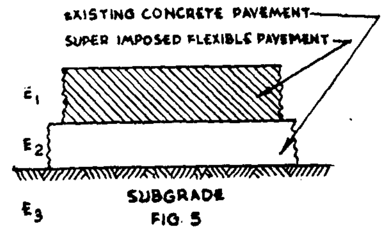


FIG. 5

But the introduction of a bituminous layer on the pavement upsets the theory of layered system adopted by Burmister which is based on the assumption that $E_1 > E_2 > E_3$ where E_1, E_2 and E_3 are modulii of elasticity for the pavement, the base course and the subgrade respectively. In the case of a superimposed layer of concrete (E_1) over a concrete pavement (E_3) having a bituminous layer (E_2) between the two, we have $E_1 > E_2 < E_3$.

Similarly if the superimposed layer consists of a flexible pavement only, Fig. 5, the layered system has $E_1 < E_2$. As Burmister's theory is inapplicable to both the cases, an approximate assessment of effective modulus of such a system has been attempted in the subsequent paragraph.

3.6. **Effect of Superimposing a Layer having Modulus of Elasticity lesser than the Modulus of the existing Pavement.**

The following is an approximate method based on the general principle laid down by Steinbrenner[2].

If an elastic layer of thickness H over a firm ground is subjected to a load which is uniformly distributed over a circular area, see Fig. 6, the maximum deformation w_m at the centre of the loaded area is

$$w_m = w_o - w_H \qquad \cdots \qquad \cdots \qquad \cdots \qquad (1)$$

where w_o = the maximum deformation at the surface of an elastic semi-infinite body, and

w_H = the maximum deformation at the depth H.

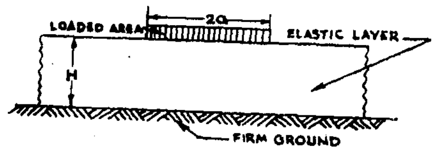


FIG. 6

By integrating the well known expression for the variation in deformation with the depth, in the case of a concentrated load (derived from Timoshenko's theory of elasticity), the deformation

$$w\varepsilon = \frac{P \times a}{E_s}\left[(1-\mu_s)\left(\frac{\varepsilon}{a} - \frac{\varepsilon^3}{a^3}\right)\sqrt{\left(1 - \frac{\varepsilon^2}{a^2}\right)}\right.$$

$$\left. + 2\left(1 - w_s^2\right)\left[\sqrt{\left(1 + \frac{\varepsilon^2}{a^2}\right)} - \frac{\varepsilon}{a}\right]\right] \quad \cdots \cdots \quad (2)$$

where ε is a point below the level of the applied load. By inserting $\varepsilon = o$ and $\varepsilon = H$ and assuming $\mu s = \frac{1}{2}$ for the sake of simplicity, we get

$$w_c = \frac{2 \times p \times a}{C} \text{ where } C = \frac{E_s}{1-\mu^2{}_s}$$

$$w_H = \frac{2 \times p \times a}{C} \times \frac{1}{\sqrt{\left(1 + \frac{H^2}{a^2}\right)}}$$

Substituting the above expressions in Equation 1, we get

$$w_m = \frac{2 \times p \times a}{C} \times \left\{1 - \frac{1}{\sqrt{\left(1 + \frac{H^2}{a^2}\right)}}\right\} \quad \cdots \cdots \quad (3)$$

If C_{mean} = the effective value of C in the layered system having two layers by using the values of W_m, w_0 and w_H we obtain from Equation 1 i.e. $w_o = w_H + w_m$

$$\text{or } \frac{2 \times p \times a}{C_{mean}} = \frac{2 \times p \times a}{C_1}\left\{1 - \frac{1}{\sqrt{\left(1 + \frac{H^2}{a^2}\right)}}\right\}$$

$$+ \frac{2 \times p \times a}{C_2}\left\{\frac{1}{\sqrt{\left(1 + \frac{H^2}{a^2}\right)}}\right\} \quad \cdots \cdots \quad (4)$$

where C_1 and C_3 relate to layer 1 and 2 respectively.

In the case of a three-layer-system having $E_1 > E_2 < E_3$ the effective mean value $E_{2\,mean}$ is obtained from the following Equation, derived in the same way as Equation 4.

$$C_{2\,mean} = \frac{C_3}{\frac{C_3}{C} - \sqrt{\left\{\frac{1 + \frac{H^2}{a^2}}{1 + \frac{\varepsilon^2}{a^2}} \times \left(\frac{C_3}{C_2} - 1\right)\right\}}} \quad \cdots \cdots \quad (5)$$

where the thickness of layers are H_1 $(H_2 - H_1)$ and ∞, Fig. 7.

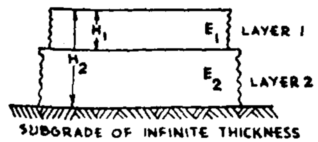


FIG. 7

The calculation thereafter can be continued by applying Burmister's theory (because $E_1 > E_{2\,mean}$ with the modulii of elasticity E_1 and $E_{2\,mean}$ and h_1 = thickness of the upper layer.

4. THE THEORY OF ELASTIC FOUNDATION (WESTERGAARD)

4.1. The assumptions and the formulae given by Westergaard are given in Appendix 2.

4.2. Stress in Concrete Pavement due to Interior Load

From the result of tests carried out by Teller and Sutherland [12], it is found that Westergaard's modified formula for interior loading is in close agreement with the test results calculated on the basis of elastic theory.

The application of Westergaard's formulae for interior load to layers of concrete superimposed over an existing concrete pavement is given in example 3.

Example 3:

A concrete pavement 8 in. thick, having a modulus of elasticity, $Ep = 3 \times 10^6$ lb per sq. in. is in contact with a subgrade having a modulus, $E_s = 3{,}000$ lb per sq. in. Assume Poisson's ratios for concrete and for subgrade to be 0.15. The flexural strength of concrete is 750 lb per sq. in. and design factor is 1.5 for runways and 1.33 for taxi tracks. Find the wheel load that can be supported by a runway pavement when the area of contact of the wheel load is 15 in. radius.

$$k=\frac{P}{w}=\frac{p\times E_s}{1.18\times p\times a}$$ by substituting the value of w from Equation 2 (a) Appendix 1).

$$=\frac{E_s}{1.18\times 15}=\frac{3000}{1.18\times 15}=\frac{3000}{17.7}=170.0 \text{ lb per in.}^3$$

for 8 in. slab if $k=170$ lb per in.^{3}, $l=30$ inches.

As $a > 1.724\ h$, $b=a$

$$\therefore \frac{l}{b}=\frac{30}{15}=2.00$$

The design stress in the case of a runway slab is 500 lb per sq. in. (taking 1.5 as design factor).

For maximum tensile stress at the bottom of the slab (S_i') using Westergaard Centre case formula for Poisson's ratio of 0.15.

$$S_i=\frac{0\cdot3162\ P}{h^2}\left[4\log_{10}\left(\frac{l}{b}\right)+1.0693\right] \quad \ldots \quad \ldots \quad (6)$$

$$\text{or } P=\frac{500\times 8\times 8}{0.3162\ (4\log(2.00)+1.0693)}$$

$$=\frac{500\times 64}{2.4781\times 0.3162}$$

$$=40{,}830 \text{ lb.}$$

and the contact pressure $p=57.7$ lb per in.^{2} Graphs[14] showing the relative values of l, k and h based on Westergaard's equations are shown in Plate I.

Example 4:

If a new layer of concrete 8 in. thick is superimposed over the existing pavement, as in Example 3, what will be the total load carrying capacity, Fig. 8.

Assuming that the state of stress in the layered system is the same as in a homogeneous system satisfying Boussenesq equation, we have:

$$\frac{E_s}{E_2}=\frac{1}{1000} \quad \therefore F_w=0.2. \quad \text{Fig. 1, Appendix 1.}$$

E for the assumed homogeneous layer which is equivalent to the two layers having modulii of elasticity as E_2 and $E_s=\frac{E_s}{F_w}=\frac{3000}{0.2}$ $=15{,}000$ lb per sq. in.; where E is the mean effective modulus of elasticity of the two layers. If $E_1=E_2=3{,}000{,}000$ lb per sq. in. the value of P is worked out below:

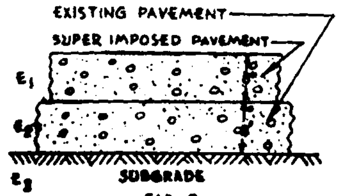


FIG. 8

For $a=15$ in., $h=8$ in., we get

$$k=\frac{15{,}000}{1.5\times 15}=667 \text{ lb per in.}^3$$

Since $a=15$ in. and $h=8$ in., $a>1.724\ h$,

therefore $b=a=15$.

From Plate I for $k=667$ and $a=15$, l is 21.0 in. and $l/b=1.4$.

$$\therefore P=\frac{500\times 64}{0.3162\ (4\log_{10}(1.4)+1.0693)}$$

$$=61{,}160 \text{ lb.}$$

and $p=86.7$ lb per sq. in.

In accordance with Westergaard's interior loading formulae, assuming that the superimposed layer rests continuously over the existing pavement, the nett increase of loadbearing capacity of the system is approximately 50 per cent more than the load-bearing capacity of the existing pavement, if the modulus of elasticity of concrete in the superimposed layer be the same as that of the existing concrete layer.

4.3. Stress in Concrete Pavement due to Corner Loading

It is generally found that a safe wheel load that could be placed in the interior of a slab might be 4 to 6 times greater than safe wheel load that can be supported at the corner. Corner loading, in majority of cases, determines the design of a concret slab.

But there are various theories for the assessment of the maximum stress in a concrete slab subjected to a corner loading and results are conflicting. Westergaard's original formula written in simplest terms using value of $u = 0.15$ is

$$S_c = \frac{3P}{h^2}\left[1 - \left(\frac{a}{l}\right)^{0.6}\right] \quad \cdots \quad (7)$$

This was modified later on to the following form by Teller and Southerland[18]

$$S_c = \frac{3P}{h^2}\left\{1 - \left(\frac{a}{l}\right)^{1.2}\right\} \quad \cdots \quad (8)$$

The modified formula is based on the assumption that, under certain conditions, the subgrade support may be somewhat less effective near the slab corner, so that the slab corner may be developing only a portion of the effective subgrade modulus, thereby utilising a somewhat smaller value of k. On the other hand, Goldbeck has taken the worst case of all, by assuming a condition when a slab is subjected to wheel load with no subgrade support, so that the load is considered to act on a cantilever of uniform strength. According to Goldbeck.

$$S_c = \frac{3P}{h^2} \quad \cdots \quad (9)$$

Sprangler[15] carried out tests on five experimental slabs to determine stresses in corner regions. Results were published only for three slabs (Nos. 2, 4 and 5). He found that the stresses calculated from Westergaard's formula were in fairly good agreement for slab No. 4 and 5 while slab No. 2 was in conformity with Goldbeck's formula.

The strains observed by the Road Research Laboratory, England[13] in the corner loading tests show closer agreement with Goldbeck's formula than with Westergaard's original formula for corner loading, Equation 7.

The following example is worked out for a corner loading to illustrate the effect of a superimposed layer on an existing concrete pavement.

Example 5 : Safe corner load on the existing pavement of Example 3

(*a*) **Westergaard's original corner loading case**

Assume $K = 170$ lb per in.^{3}, $a = 15$ in. and

$h = 8$ in. (as previously obtained)

$b = a = 15$ in,

$\therefore\ l = 30$ in. $a/l = 0.5$.

Substituting these values in Equation 7, we get

$P = 27{,}510$ lb and $p = 39.19$ lb per sq. in.

(*b*) **Westergaard's revised corner loading case**

Substituting the above noted values in Equation 8 we get $P = 18{,}890$ lb and $p = 26.7$ lb per sq. in.

(*c*) **Goldbeck's formula**

Substituting the values in Equation 9, we get $P = 10{,}700$ lb and $p = 15.2$ lb per sq. in.

Example 6 : Safe corner load on the superimposed pavement of Example 4

We have $a = 15 = b$ and $l = 21$ in.

$a/l = 0.72$ and the load bearing capacity of the whole system is given below.

(*a*) **Westergaard's revised corner loading case**

Substituting the values in Equation 8, we get :

$P = 32{,}800$ lb and $p = 45.2$ lb per sq. in.

(*b*) **Goldbeck's formula**

Substituting the values in Equation 9, we get

$P = 10{,}700$ and $p = 15.2$ lb per sq. in.

In other words, according to Goldbeck's formula, there is absolutely no increase in the load bearing capacity if a new layer of concrete is laid over an existing layer.

5. FLEXIBLE PAVEMENT AS A SUPERIMPOSED LAYER

5.1. A bitumen-aggregate mixture consists of a solid granular material (aggregate), a liquid (bitumen) and a gas (air). It would therefore be correct to assume that a bituminous pavement forms a system analogous to that of soil. The difference is that the voids in the aggregate of bitumen mixture are filled with a highly viscous bitumen, whereas the voids in the soil are filled with water which has low viscosity. Deformation of the pavement gives rise to resistance due to the high viscosity of bitumen. In case of settlement, the extrusion of excess liquid from the voids of the aggregate takes a very long time in bitumen-aggregate mixtures as the viscosity of bitumen is about 10^7 to 10^9 times greater than that of water.

Plasticity as a factor in the design of dense bituminous mix can be evaluated[18]. Its value is dependent on two factors, viz, (a) void factor (an expression showing the influence of the void-content on the plastic properties) and (b) the F.B. factor (filler to bitumen factor) of the mixture. Assuming that no plastic flow occurs, a bituminous pavement is treated as an elastic layer.

5.2. The Effect of a Superimposed Bituminous Layer over an Existing Concrete Pavement

It is necessary, for the purpose of making comparative study of a bituminous and rigid overlays, to evaluate the supporting capacity of an existing concrete pavement with a superimposed bituminous layer. The method adopted which assumes that the bituminous layer has elastic properties is illustrated by example 7.

Example 7:

Assume a 6 in. bituminous layer over an existing concrete pavement 8 in. thick having the modulus of elasticity of 3×10^6. The value of E for subgrade is assumed to be 3×10^3, and that of bituminous layer as 5×10^4.

The maximum permissible deflections in pavements accepted by Burmister[4] are given below:

(i) bituminous pavement 0.2 in.

(ii) concrete pavement 0.05 in.

The maximum deflection at the centre of the load application, according to Steinbrenner, is given by Equation 1, i.e. $(0.2-0.15)=0.15$ inch.

Assuming 0.15 in. as the maximum deflection under the load, the value of the wheel load P has been worked out with the help of Burmister's formula thus —

$$\frac{E_s}{E_2}=\frac{3\times10^3}{3\times10^6}=\frac{1}{1000}\,;\ a=15 \text{ inch.}$$

$$\therefore F_w \text{ from (Fig. 1, Appendix 1)} = 0.20$$

$$\therefore \text{ Modified value } E_s=\frac{3000}{0.20}=15{,}000 \text{ lb per sq. in.}$$

(Example 4 for explanation).

$$\frac{E_s \text{ modified}}{E_1}=\frac{15{,}000}{50{,}000}=\frac{1}{3.3} \text{ from Burmister's}$$

curve (Fig. 1, Appendix 1).

$$h=\frac{6}{15}\times a= .4\,a.$$

$F_w = 0.9$. Substituting this value in Equation 2, Appendix 2.

$$w=\frac{1.5\times p\times a\times F_w}{E_s}$$

$$\therefore\ p=\frac{.15\times15{,}000}{15\times1.5\times0.9}$$

$$=111.0 \text{ lb per sq. in.}$$

$$P=78493 \text{ or } 78{,}000 \text{ lb.}$$

This result has been obtained by assuming that the bituminous layer is elastic. It is, therefore, necessary to compare the value thus obtained with the normal method of designing bituminous pavements, say, by the CBR design curves.

5.3. Application of CBR Design Curves

The California Bearing Ratio (CBR) method is a semi-empirical one. So a correct correlatien between the analytical methods of Burmister and Westergaard and CBR method is not practicable. Nevertheless, many attempts have been made to establish approximate correlation between CBR and K values to find out what thickness of a flexible pavement could have been used, if an existing concrete pavement is replaced by an equivalent base course having the same CBR value. An attempt has been made merely to determine the thickness of a flexible overlay with the help of CBR method for comparison with the result obtained in Example 7.

The approximate values of CBR corresponding to the various values of k given below have been published by Portland Cement Association, U.S.A.

K values	*C.B.R.*
100	3
200	10
250	20
500	50
800	100

The following Table given by Wilson and Williams [23] gives the relationship between k, CBR and E_s for different types of soils. The values of k given are for pressures required to cause a settlement of 0.05 in. using a 15 in. radius plate for the test. A deflection of 0.00325 in. in the case of a disc with a 0.97 in. radius (such as is used in the CBR test) corresponds to the standard deflection of 0.05 in. for the plate bearing test with a 30-in. plate. This value is very close to the origin of the load-deflection curve in the CBR test. Hence the slope of the initial tangent is taken as equal to the approximate value of p/w. Assuming a parobola through the origin and the standard CBR pressures for deflections of 0.1 in. and 0 2 in., the slope of the tangent has been found to be equal to 30,000 lb per inch cube; hence $E_s=409$ CBR and $k=19.5$ CBR.

Description	*Shear strength*	*k lb. per in.*^{3}	*C.B.R.*	E_s *lb. per sq. in.*
Soft clay	$c = 400$	41	2.1	858
Stiff clay	$c = 2000$	205	10.5	4,300
Loose sand	$\phi = 32°$	230	to	4,800
Dense sand	$\phi = 38°$	500	50	10,500

This correlation has been derived solely for the comparison of pavement thicknesses obtained by various methods. It is not suggested for use in the design of pavements.

5.4. The Effect of a Flexible Overlay on a Rigid Pavement

It has been shown in Example 13 para. 7.3, that a 6 in. thick bitumen overlay can take a heavier wheel load than a 8 in. concrete overlay having the same modulus of elasticity as that of the existing pavement. This is due to:

(a) the larger maximum deflection that is permissible in flexible pavement than in the rigid pavement;

(b) the flexible pavement being free from temperature stress which takes away a large portion of the load bearing capacity of a rigid pavement.

6. TEMPERATURE STRESS

6.1. In a rigid pavement temperature stress is caused by changes in volume consequent on changes in temperature. Sources of heat changes in a pavement are :—

(a) the sun by radiation,

(b) the air by contact, and

(c) the subgrade or base below by contact.

Change in the amount of heat gives rise to temperature variations in the pavement causing displacements. Since these displacements are partly prevented, temperature variations cause stresses which in many cases can be of the same magnitude as wheel load stresses. It is necessary to determine the tensile stresses which are produced at the bottom of the pavement due to temperature variations and add them to the wheel load stresses. As unreinforced concrete pavements have been considered in this Paper, the stresses in the upper portion of the pavement are not of importance.

6.2. Since pavements are provided with expansion and contraction joints, the stress due to a temperature rise is governed by the resistance to friction between the pavement and the subgrade during the expansion of the pavement. Accordingly we can divide the effect of temperature changes into two parts, viz :

(a) the effect of the temperature variation between the top and the bottom surfaces of the pavement, the mean temperature changes in a cross section being zero.

(b) The effect of uniform temperature change in the whole cross section.

The latter gives rise to friction between the pavement and the subgrade, and these forces can be calculated when the coefficient of friction is known.

6.3. Temperature Gradient

When the temperatures at the top and the bottom surfaces of a concrete pavement are not the same the pavement is subjected to "temperature differential", the magnitude of which at any time being the difference between the temperature at the top and bottom. During the day the pavement delays transmission of surface heat to the bottom while at night the top surface looses heat more rapidly than the bottom. This produces reversal of stress in the pavement every twenty-four hours [10].

A typical distribution of temperature known as 'temperature gradient curve' in a pavement is shown in Fig. 9.

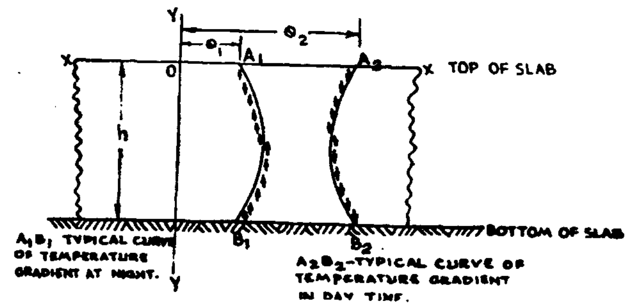


Fig. 9. Temperature gradient

The depth of the slab is shown as ordinate counted downwards and the temperature as abscissa. At any time t_1 the distribution of temperature in the slab from the top to the bottom surface of the pavement is represented by the curve A_1B_1. The temperature distribution at a time t_2 is shown by the curve A_2B_2 and the direction of the flow of heat is shown by arrows. The area $A_1B_1B_2A_2$ included between the two curves gives a measure of the quantity of heat 'f' given out or absorbed by the pavement between the time t_1 and t_2 and is given by the equation

$$f = \int_0^h (\theta_2 - \theta_1)\, dh \qquad \dots \qquad (10)$$

The exposed surface temperature of a pavement is influenced more by the intensity of sun's rays than by actual air temperature.

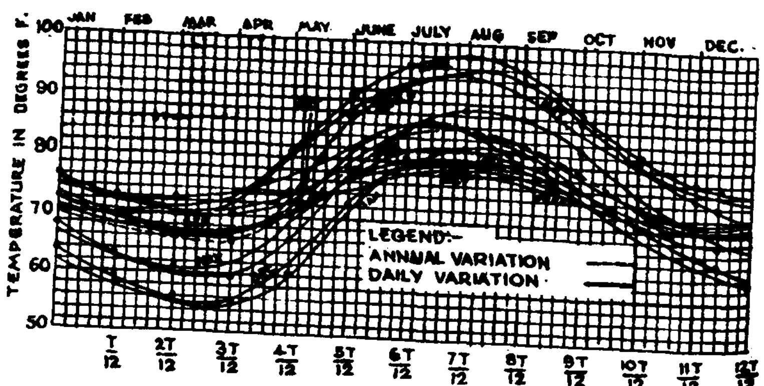


Fig. 10. Variation of diurnal and annual temperature at Poona,

6.4. Air-temperature Variations

Typical annual and diurnal variations of average air temperatures in Poona obtained from the Meteorological Office, Poona, are shown in Fig. 10. The values are shade temperatures and represent the averages of observations over a number of years. The curves are not unlike sine curves.

6.5. Surface-temperature Variations

Westergaard in his earlier computations of temperature stresses assumed a linear temperature gradient. But from actual stress measurement by Thomlinson [22] it is found that the temperature gradient is not, a straight line.

The temperature gradients obtained from observations in the Arlington tests and temperature gradients calculated from Thomlinson's values are shown in Fig. 11. Comparing corresponding curves it will be seen that the experimental curves have a gradient inclined towards a higher surface temperature than the calculated results. Therefore, the Arlington tests show that modification of stresses calculated by Thomlinson is necessary.

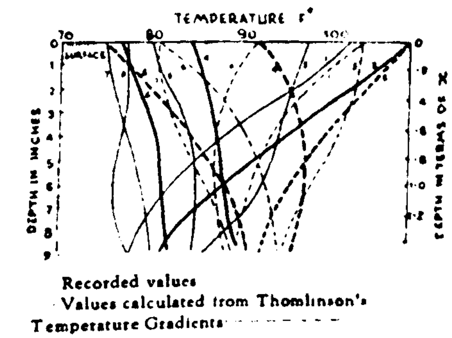


Fig. 11. Temperature gradients for a concrete slab in summer. Arlington, U.S.A.

Westergaard took into consideration this factor and modified his earlier formula.

Movement of slabs in a concrete pavement due to temperature variations, when superimposed by another concrete layer with or without a bituminous carpet between the two slabs have been studied at the Engineer Research Wing of the College of

Military Engineering, Kirkee, Poona. Figs. 12 and 13 are plotted from the observations. It is seen there is very little change in the behaviour of the bottom concrete slab when a bituminous carpet is introduced.

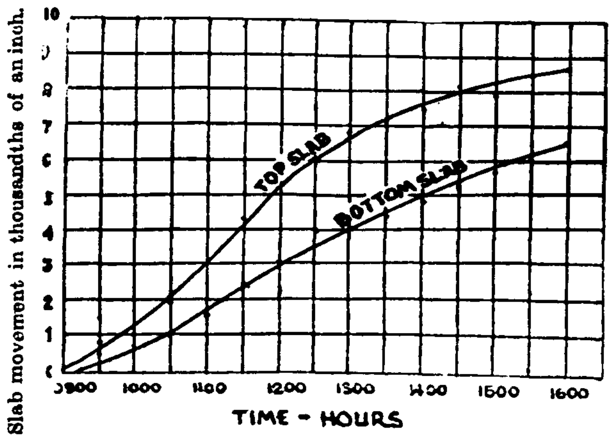


Fig. 12. Movement of concrete slab with no separating media between two slabs.

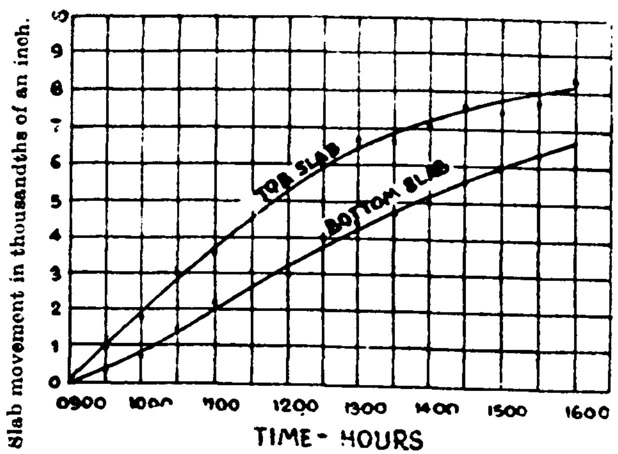


Fig. 13. Movement of concrete slabs when a separating layer of bitumen 80/100 and sand is applied at 8 lb per 5 sq. ft.

In the calculation of temperature stress in a concrete pavement superimposed by a concrete overlay with a bituminous carpet between the two pavements, the surface temperature

variation curve has, therefore, been assumed to be the same when there is no bituminous carpet between the two pavements.

6.6. Relation between Temperatures of Air and Surface of Pavement

There is a definite relationship between air and surface temperatures where the air-temperature cycle repeats itself several times in the same way [16].

Data on surface-temperature variations and their relation with air-temperature variations under Indian conditions are not available. As the diurnal variations of average temperatures at Poona are almost regular and even, Fig. 10, it has been assumed in subsequent calculations that the ratio of amplitude of air-temperature variation for a rough concrete slab in India is the same as the values observed in U.K. The relation between surface and air temperatures prepared by Champion[16] is reproduced in Table 2.

TABLE 2

Relation between Surface and Air Temperatures*

S. No.	Type of material & surface treatment	Amplitude of surface temperature variation / Amplitude of air temperature variation: Maximum	Average summer	Average winter
1	Smooth concrete slab as laid ..	3½ to 4	1.5-1.8	0.9-1.0
2	Smooth concrete slab treated with silicate ...	3 to 3½	1.2-1.4	1
3	Rough concrete slab as laid ...	4	1.6-2	0.8-1.0
4	Concrete slabs with surface darkened ...	4 to 4½	1.8-2	1.0-1.1
5	Concrete slab with the surface concrete of tarmac or asphalt ...	4 to 4½	1.8-2	1.0-1.1
6	Tarmac on hard core ...	4½ to 5	2	0.9-1.0

* S. Nos. 1 and 4 were obtained from readings at Colnbrook recorded by the Road Research Laboratory, U. K. and items 2, 3 and 6 from observations at Deopham Green.

6.7. Thomlinson's Analysis

This analysis is based on the assumption that the temperature is uniform on any place parellel to the exposed surface. Then the stresses can be divided into 3 parts, due to:

(a) Non-linear variation of temperature in a direction normal to the exposed surface.

(b) An overall temperature change where the consequent change in length is resisted.

(c) A mean, uniform temperature gradient normal to the exposed surface where the resulting warping is resisted.[22]

The daily variation in air temperature in Poona can be assumed to be approximately sinusoidal, Fig. 10. Its effect can therefore be studied with the help of the expression for temperature stress obtained by Thomlinson's analysis which is based on the assumption that the surface temperature varies according to a simple harmonic law.

The stress due to temperature are

(a) internal stress $= A_i \text{ Sin } \frac{2\pi}{T}(t + t_i)$ ... (11)

(b) stress due to end restraint $= A_e \text{ Sin } \frac{2\pi}{T}(t + t_e)$, and ... (12)

(c) stress due to warping restraint $= A_w \text{ Sin } \frac{2\pi}{T}(t + t_w)$ ... (13)

based on the assumption that surface temperature variation is sinusoidal so that $\theta'_0 = \theta_0 \frac{\sin 2\pi t}{T}$ where t is the time at which θ'_e is to be calculated and T is the periodic time of viz. day or 1 year in a seasonal analysis. The temperature at any depth 'x' from the top surface of a concrete slab is evaluated from the equation

$$\theta x = \theta_0 e^{-\frac{x}{h}\sqrt{\frac{\pi}{T}}} \sin\left(\frac{2\pi}{T}t - \frac{x}{h}\sqrt{\frac{\pi}{T}}\right)$$

where θx denotes temperature on a plane distant x from the surface at a time t.

h^2 = diffusivity of concrete

$= \frac{\text{thermal conductivity}}{\text{heat capacity per unit volume}}$

= 0.009 C.G.S. units.

For temperature at the bottom face of a concrete slab $x = d$, and the function $\frac{d}{h}\sqrt{\left(\frac{\pi}{T}\right)}$ is denoted by $\propto$. The values of A_i, A_e and A_w and t_i, t_e and t_w are obtained from design charts for internal stress, end-restraint stress and warping restraint stress from the values of $\propto$.

The values thus obtained from design curves for stresses due to end-restraint and warping restraint assume that the restraints are complete. In practical cases, the restraint in both cases will be partial except in long slabs. Thus two now factors, viz., K_e and K_w which are usually less than unity are introduced.

The resultant stress due to temperature expressed by Thomlinson is

$$A_i \sin \frac{2\pi}{T}(t + t_i) + K_e \cdot A_e \sin \frac{2\pi}{T}(t + t_e) + K_w \cdot A_w \sin \frac{2\pi}{T}(t + t_w)$$

The coefficient of warping-restraint can be found from Fig. 14 given by Bradbury.

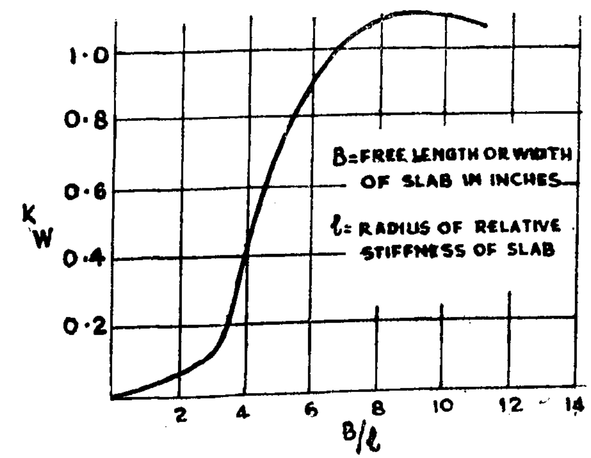


Fig. 14 Warping stress coefficients—Bradbury's Formula.

It will be seen that a knowledge of both temperature differential and the amplitude of surface temperature variation is required for deriving the values of temperature stresses according to Thomlinson's analysis. The correlation between the ratio of temperature differential to temperature amplitude for various depths in a concrete slab is shown in Fig. 15.

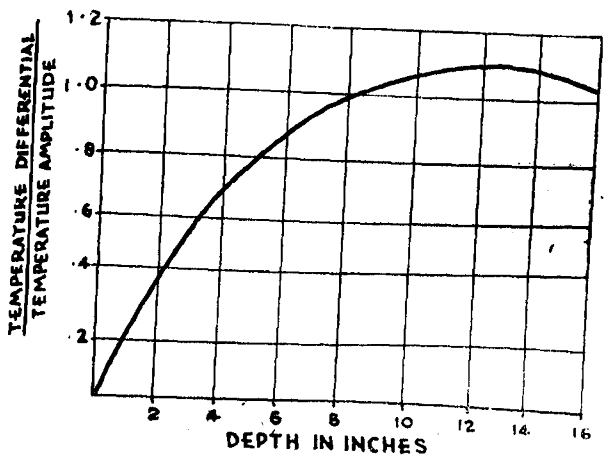


Fig. 15. Ratios of temperature differential to temperature amplitude for various thicknesses of concrete slab

6.8. Westergaard's Analysis

The expression given by Westergaard for temperature stress based on the linear temperature gradient for interior loading is:

$$S_t = K_w \frac{Eet}{2} \qquad \cdots \quad (14)$$

when S_t is the maximum tensile stress equal in all directions.

Westergaard modified this expression to include the co-efficient of warping restraint and obtained the following equation:

$$S_t = \frac{Eet}{2} \times \frac{(K_{w1} + K_{w2})}{1-\mu^2} \qquad \cdots \quad (15)$$

where K_{w1} refers to the breadth of the slab and K_{w2} to the length of the slab. Both co-efficients are obtained by the method suggested in para. 7.2.

6.9. Conclusion

Detailed investigation into temperature stresses in concrete under the various climatic conditions obtaining in India has not yet been made. It cannot, therefore, be said with certainty which of the two methods of computation is applicable to Indian conditions. Judging from the studies made by the Department of Scientific and Industrial Research in the U. K. it seems that Thomlinson's method is likely to give a closer approximation to the actual state of stress than the method adopted by Westergaard.

7. COMBINED EFFECT OF TEMPERATURE AND WHEEL LOAD

7.1. The combined effect of temperature stress and stress due to wheel load has been illustrated by examples 8 to 13. The following procedure has been adopted for working:

(*a*) Determine the stresses due to temperature in an overlay.

(*b*) Deduct the value of temperature stress so obtained from the safe working stress in concrete. Assume that the balance is available for supporting static load.

(*c*) Keeping the area of contact of the static load, subgrade conditions and the strength of concrete constant, work out the maximum wheel load that the overlay can support for the balance of stress obtained from (*b*).

7.2. Computation of Temperature Stress

Example 8:

Find the temperature stress in a 8-in. concrete slab in Poona resting on a compacted subgrade having $E_s = 3{,}000$ lb per sq. in.

From Fig. 10 it is found that the maximum diurnal temperature variation in Poona is 28° F. Therefore the amplitude of surface air temperature variation is 14°. Amplitude for surface temperature of concrete is $14 \times 1.6 = 22.4°$ F, Table 2 item 3. For 8-in. concrete slab, the ratio temperature differential/ temperature amplitude is 0.96, Fig. 15. Therefore temperature differential $= 22.4 \times 0.96 = 21.5°$ F.

The value of $k = 170$ lb per in.^{3} (see para. 4.2, Example 3)
$l = 29.8$ in.

Assuming a slab 15 ft × 20 ft, from Fig. 14

$$\frac{B_2}{l} = \frac{240}{29.8} = 8.06 \quad K_{w2} = 1.1$$

and

$$\frac{B_1}{l} = \frac{180}{29.8} = 6.05 \quad K_{w1} = 0.9$$

(a) Solution by Westergaard's method of analysis :

Assume e = thermal coefficient of linear expansion of concrete $= 3 \times 10^{-6}$ per degree F.

E = modulus of elasticity of concrete $= 3 \times 10^{6}$ lb per sq. in.

(i) warping stress in the interior of the slab (Equation 10, Appendix 2)

$$= \frac{3 \times 10^{-6} \times 3 \times 10^{6} \times 21.5}{2} \times \frac{0.9 + 0.15 \times 1.1}{1 - (0.15)^2}$$

$$= \frac{193.5}{2} \times \frac{1.065}{0.977} = 105.5 \text{ lb per sq. in.}$$

(ii) warping stresses along the edge of the slab (Equation 11, Appendix 2)

$$= \frac{K_w \times E \times e \times t}{2}$$

$$= \frac{1.1 \times 3 \times 10^{6} \times 3 \times 10^{-6} \times 21.5}{2}$$

$$= 106.5 \text{ lb per sq. in.}$$

(iii) warping stress along the diagonal (Equation 12, Appendix 2)

$$= \frac{Eet}{3(1-\mu)} \times \sqrt{\frac{a}{l}}$$

$$= \frac{3 \times 10^{6} \times 3 \times 10^{-6} \times 21.5}{3(1 - 0.15)} \times \sqrt{\frac{15}{29.8}}$$

$$= \frac{9 \times 21.5}{3 \times 0.85} \times 0.7 = 53 \text{ lb per sq. in.}$$

(b) Adopting Thomlinson's method of analysis using Thomlison's notation, we get

$$\propto = \frac{d}{h} \times \sqrt{\frac{\pi}{T}} = \frac{8 \times 2.54}{.095} \times \sqrt{\left(\frac{\pi}{24 \times 60 \times 60}\right)} = 1.296.$$

again $\dfrac{Eet}{1-\mu} = \dfrac{3 \times 10^{6} \times 3 \times 10^{-6} \times 21.5}{(1 - 0.15)} = 230$

$\therefore$ From equation 15 we get maximum temperature stress = 138 lb per sq. in.

Example 9 :

Find the temperature stress in a 16-in. a concrete slab on a compacted subgrade in Poona. Working on the same condition as in Example 8, we get

$$\frac{\text{Temperature differential}}{\text{Temperature amplitude}} = 1.1, \quad \text{Fig. 15}$$

$\therefore$ Temperature differential $= 1.1 \times 22.4 = 24.64$ degrees F.

Now $k = 170$ lb per sq. in.[3] Therefore, for a 16-in. slab $= 50$ inches.

$$\left.\begin{aligned} \frac{B_2}{l} &= \frac{240}{50} = 4.8\,; \quad k_{w2} = 0.7 \\ \frac{B_1}{l} &= \frac{180}{50} = 3.6\,; \quad k_{w1} = 0.1 \end{aligned}\right\} \text{Fig. 14}$$

(a) **Adopting Westergaard's analysis**

(i) warping stress in the interior slab

$$= \frac{3 \times 10^{-6} \times 3 \times 10^{6} \times 24.64}{2} \times \frac{0.1 + 0.7 \times 0.15}{1 - (0.15)^2}$$

$$= \frac{221.76}{2} \times \frac{.205}{.975} = 24 \text{ lb per sq. in.}$$

(ii) warping stress along edge of the slab

$$= \frac{0.7 \times 3 \times 10^{6} \times 3 \times 10^{-6} \times 24.64}{2} = 77.5 \text{ lb per sq. in.} \sqrt{\frac{16}{50}}$$

(iii) warping stress along diagonal of the slab

$$= \frac{3 \times 10^{6} \times 3 \times 10^{-6} \times 24.64}{3 \times 0.85} \times \sqrt{\frac{16}{50}}$$

$= 49$ lb per sq. in.

(b) Applying Thomlinson's method of analysis (using Thomlinson's notations) having $h^2 = 0.0009$ C. G. S. unit

$$\propto = \frac{d}{h} \times \sqrt{\frac{\pi}{T}} = \frac{16}{0.095} \times 2.54 \sqrt{\left(\frac{\pi}{60 \times 60 \times 24}\right)}$$

$$= 2.592$$

$$\frac{Eet}{1-\mu} = \frac{3 \times 10^{6} \times 3 \times 10^{-6} \times 24.65}{0.85} = 262$$

$\therefore$ Equation 14 gives maximum temperature stress (Tensile) = $262 \times 0.532 = 129$ lb per sq. in. (The value of 0.532 is obtained from Reference 22).

7.3. Computation of Combined Temperature and Wheel Load Stress

Taking flexural strength in a concrete pavement = 750 lb per sq. in. and design factor = 1.5, the working stress in concrete (flexure) is equal to 500 lb per sq. in. In the succeeding examples, evaluation of allowable wheel loads has been made after deducting from the working stress in concrete the values of maximum temperature stresses according to Thomlinson's analysis.

Using Westergaard's centre case formula with Poisson's ratio for concrete = 0.15.

$$s_i = 0.31625 \frac{P}{h^2} \left(4 \log_{10} \frac{l}{b} + 1.0693\right)$$

$$\therefore P = \frac{s_i \times h^2}{0.31625 \times \left(4 \log_{10} \frac{l}{b} + 1.0693\right)}$$

Example 10:

Thickness of slab, $h = 8$ in ; $k = 170$ lb per in.^{3}, $l = 29.8$ in. as in Example 8. As $1.724h = 1.724 \times 8 = 13.792$ is $> a$.—Fig 16.

$\therefore b = a = 15$ in. Maximum temperature stress = 138 lb per sq. in. (see Example 8).

$\therefore$ available flexural strength to take up wheel load stress after catering for temperature stress = $500 - 138 = 362$ lb per sq. in.

$\therefore P =$ allowable wheel load

$$= \frac{362 \times 64}{0.31625 \times \left(4 \log_{10} \frac{29.8}{15} + 1.0693\right)}$$

$$= \frac{362 \times 64}{0.31625 \times 2.617}$$

$= 3.,000$ lb.

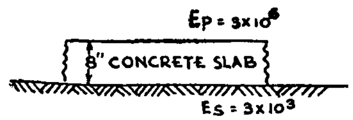


FIG. 16

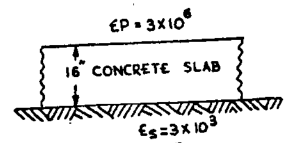


FIG. 17

Example 11:

$h = 16$ in.; $k = 170$ lb per in.^{3}; $l = 50$ in. Fig. 17.

$1.724h = 1.724 \times 16 = 27.584 > a$.

$$\therefore b = \sqrt{(1.6 \times 15^2 + h^2} - 0.675h)$$

$$= \sqrt{\{1.6 \times 15^2 + (16)^2} - 0.675 \times 16\}$$

$= 14.1$

Maximum temperature stress = 129 lb per sq. in., Example 9

$\therefore$ Flexural strength available to take wheel load = $500 - 129$ = 371 lb per sq. in.

$\therefore P$ = allowable wheel load.

$$= \frac{371 \times 256}{0.31625 \left(4 \log_{10} \frac{50}{14} + 1.0693\right)}$$

$$= \frac{371 \times 256}{0.31625 \times 3.09}$$

$= 99,000$ lb.

Example 12:

An 8 in. concrete slab overlay on an existing concrete pavement of 8 in. thickness, Fig. 18.

Applying Burmister's method of solution for layered system :—

For (2) and (3) layers,

$$\frac{E_s}{E_2} = \frac{3 \times 10^3}{3 \times 10^6} = \frac{1}{1000}$$

or $F_w = 0.2$

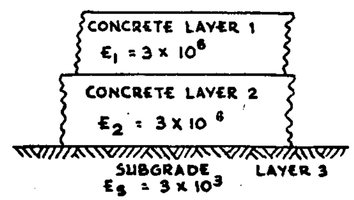


FIG. 18

Reinforced value of E_s due to overlay (see Examples 4 and 7)

$$= \frac{3000}{0.2} = 15{,}000 \text{ lb per sq. in.}$$

$\therefore$ Equivalent k for (2), (3) layers, from Equation 3, Appendix 2

$$= \frac{15000}{1.18 \times 15} = 846 \text{ lb per in.}^3$$

$\therefore \quad l = 19.3$ (from Plate I).

Applying Westergaard's method of analysis for layer 1 (the flexural strength available for wheel load is 371 lb per sq. in as in Example 11).

$$P = \frac{371 \times 64}{0.31625 \times \left\{4 \log_{10}\left(\frac{19.3}{15}\right) + 1.0693\right\}}$$

$$= 49{,}500 \text{ lb.}$$

Example 13:

Bituminous Overlays

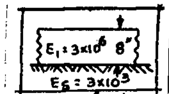
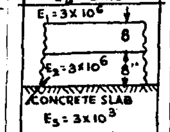
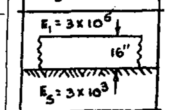
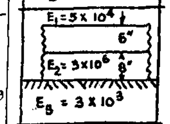
In this case the thickness of a flexible pavement has been worked out based on the assumption that the existing concrete pavement acts as a base course and is free from temperature stress. The C.B.R. values have been determined from the correlation [20] given in para. 5.3.

The present case is similar to Example 4 except that the superimposed concrete layer has been replaced by a bituminous superimposed layer. The value of k, already worked out in Example 4 is 667 lb per in.^{3}. The corresponding value of C.B.R. is 75 (obtained from the co-relations given in Para. 5.3.). Using a single wheel load having a tyre pressure of 100 lb per sq. in., the designed thickness of the flexible pavement is about 6 in. to support a wheel load of 78,000 lb.

Results obtained by combining wheel load and temperature stresses for Examples 10 to 13 are tabulated in Table 3.

TABLE 3

Temperature and Wheel Load Stresses on different types of Pavement and Overlays

Specification of pavement	Sketch	Thickness of slab in.	Temperature stress lb per sq. in.	Allowable whele load stress—lb	Wheel load lb
Cement concrete 8 in. on compact subgrade, Example 10.		8	138	362	33,000
8-in. cement concrete overlay on existing 8 in. concrete slab on a compact sub-grade, Example 12.		16	129	371	49,500
16-in. cement concrete on compact subgrade example 11.		16	129	371	99,000
6-in. bitumen concrete on 8 in. cement concrete resting on compact subgrade, Example 13.		14	...	...	78,000

7.4. **Summary of Results obtained by combined Temperature Stress and Wheel load Stress**

The values in Table 3 show that for the given temperature conditions, a sixteen inch thick monolithic concrete slab has a loadbearing capacity three times more than an eight-inch concrete slab. On the other hand, if an existing eight-inch concrete slab is superimposed by another eight-inch concrete slab having the same modulus of elasticity, the net increase in the load bearing capacity is only one and half times that of the existing slab. The introduction of a bituminous superimposed layer of 6 in. on an existing concrete pavement of 8 in. has the effect of increasing the load bearing capacity of the existing pavement to two and a half times the original value.

It is evident from the comparative values given above that a flexible overlay on a rigid pavement is an economical method of strengthening. The concrete pavement acts as a base course and, therefore, is subjected to very little temperature changes.

Hence the flexural strength of concrete is largely available to withstand the stress due to wheel load.

This study indicates the possibility of using concrete pavements not only as the immediate base but also as the sub-base since the temperature changes at the depth of the sub-base would be negligible, provided the flexible layers above the concrete slab would resist plastic deformation. Such a design can be adopted at sites where a normal type of flexible construction would necessitate a thickness of base course that is neither practicable nor economical due to the poor quality of the subgrade.

8. AIRFIELD PAVEMENTS FOR JET AIRCRAFT

8.1. The theoretical analysis of load-bearing capacities of various types of overlays given above shows that a flexible overlay of bituminous construction over an existing concrete pavement is better than other types of overlays. Bituminous construction can be repaired easily in the event of a local settlement, is impervious, and is lower in cost than a concrete construction. But in modern airfields, the use by jet aircrafts has ruled out, for the present, the use of ordinary bituminous material which cannot withstand the adverse effect of fuel spillage and exhaust heat and blast of jet engines. The spillage of paraffin is the maximum while the machine is standing with its engine running. This effect is noticeable on aprons and at the ends of runways. Hot gases at temperatures from 1600 deg. F. to 2200 deg. F. from the exhaust of jet engines impinge upon the pavement at temperature of 400 deg. F. to 500 deg. F. with a velocity of 200 to 300 ft per second. In India, unless experimental verification proves otherwise, temperature criteria may be taken as 400 deg. F. At Heathrow airfield, a tail-wheeled Viking aircraft fitted with jet engines caused pitting of about $\frac{1}{4}$ in. to the concrete after running for about 10 minutes. But the angle of jet, which is excessive in tail-wheeled aircrafts, has been considerably reduced in the newer types which are mostly nose-wheeled. A Vampire aircraft has a maximum angle of about 3 degrees and produces no effect on a concrete pavement[1], but it might affect bituminous material. Manufacturers are now busy trying to solve this difficulty and it is expected that future jet aircrafts might not present this problem at all[1].

8.2. The Use of Flexible Pavement by Jet Aircraft

In addition to fuel spillage and exhaust heat and blast, pavements are generally subjected to high pressures of 110 lb per sq. in. by modern aircraft. The effects of these are pronounced in areas where engine starting or pre-take off engine checks and engine maintenance tests are carried out, for example on runway ends. At a symposium on airfield pavements[17], the view that an airfield pavement should withstand a blast having a temperature range of 400 to 500 deg. F. and a velocity from 200 to 300 ft per second was expressed. The view of the U. S. Corps of Engineers is that the effects of the jet blast are not found to be detrimental when the aircraft moves on the runway or on the taxi track. In the investigation carried out by them, it was found that a dense-graded, hot-mixed, asphaltic concrete withstood the effect of jet exhaust from most of aircraft used in the test. Old dense-graded hot-mixed asphaltic concrete pavements have shown satisfactory resistance to jet fuel spillage as well. The above was supported by Mr. J. M. Grifith of the Asphalt Institute.

In the modernization of the runway at Ohio, a concrete overlay with slag aggregate for the runway ends and an overlay of asphaltic concrete 6 in. thick for the remaining portions of the pavement were laid.

The practice adopted by the British Air Ministry is to lay on an existing bituminous pavement a protective overlay using a protein emulsified bitumen mixed with an equal portion of Portland cement as a binder. For use at runway ends, this mixture is combined with fine aggregate but it is not well adopted for use in areas where fuel spillage occurs.

The view[17] expressed by Henry Aaron, Paving and Soils Branch, Aeronautics Administration, is that dense asphaltic concrete serves very well for resurfacing airfields which are being subjected to loads greater than those for which they were designed.

The above shows that bituminous pavements for use by jet aircraft cannot be ruled out.

8.3. Development of Bituminous Pavement to resist Jet Effect

Generally speaking, there are two ways of increasing the melting point of bitumen, viz., by blowing air whereby high melting point blown bitumen is obtained and using suitable fillers. If a filler is used with a straight bitumen emulsion e.g., Colas, the emulsion will break into water and bitumen. To prevent the emulsion from breaking while mixing with the filler, a retarding agent which will help in the colloidal suspension has to be introduced. In some of the airfields in the U. K. a special cold emulsion is used as a binder and the filler used is Portland cement. To this is added calcium chloride approximately 1/3 to

⅓ of the weight of cement so that a smooth impervious surface is obtained after rolling. In India slate powder has been in use as a filler in the manufacture of P. B. S. Slate powder alone or mixed with asbestos gives a type of filler better than that given by Portland cement. With the abundance of natural asbestos, slate and mica in India, it should be possible to find a suitable filler that might stand the deleterious effect of jet exhaust.

9. SOME RECENT TESTS

9.1. Brief summary of the tests on concrete pavements conducted at De Havilland Airfield at Hatfield [1] and on different thicknesses of concrete and bituminous overlays by the U. S. Corps of Engineers is given is Appendix 3. Even though sufficient data was not collected, the tests at the Hatfield airfield showed that there is an increase in the value of k with the use of better subgrades. The trials [26] carried out by the U.S. Corps of Engineers showed that a 9-in. flexible pavement consisting of a 6 in. lime rock base with a 3-in. asphaltic concrete surface provided the best reinforcement for a 6-in. thick concrete pavement. Also, even a thin bituminous separation course was conducive to cracking of concrete overlays.

10. CONCLUSIONS

10 1. The use of a properly designed flexible overlay appears to be the most satisfactory method of increasing the bearing capacity of a rigid pavement. But this has to be made resistant to the deleterious effects of exhaust gases from jet aircraft. Investigation under Indian conditions on this aspect of flexible pavements is desirable.

10.2. To cope with the increasing wheel load and tyre pressure, concrete pavements have to be made thick. Also, the maximum thickness to which concrete can be laid satisfactorily in one layer using modern spreading and compacting machines is limited. Therefore, a layered construction is necessary. But concrete construction in layers can be justified only if concrete in any layer has a modulus of elasticity greater than that of the layer below it. Separation of the layers with a thin course of bituminous construction has little effect in reducing the temperature stress in the bottom layer. Also, extensive trials carried out by the U.S. Corps of Engineers have shown that the bituminous separating course is conducive to cracking of concrete overlays.

The evalution of temperature stresses in a layered system of concrete pavement is based on assumptions for which data applicable to Indian conditions are not available. In general, an attempt has been made to study the effect of various factors on the load-bearing capacity of an overlay so far as wheel load and temperature stress are concerned. No attempt has been made to indicate a rational method of design of an overlay as it would mean consideration of other factors, e.g., subgrade moisture increase, fatigue of the materials constituting the pavement and subgrade, which are beyond the scope of this Paper.

10 3. Investigation regarding Temperature Stress under Indian Conditions

For the assessment of temperature stresses in a concrete pavement, data on air-temperature variations over a number of years are required. Also, a knowledge of the relationship, if any, of air-temperature variations to surface-temperature variations for various types of pavement surfaces is necessary. Although values of shade temperatures over a number of years are obtainable for various places in India from the Meteorological department, data on surface-temperature variation in relation to air-temperature variations are not availale. In fact, we have no data to show that Thomlinson's method of analysis of temperature stress is really applicable to Indian conditions. Such investigation in different regions of India, if carried out by various research laboratories established by the Central or State Governments, would be of immense value in evolving a rational design for a concrete pavement. Perhaps some sort of co-ordination of the investigation and the collation of data obtained from various regions of India by a centrally constituted authority would be desirable.

10.4. Development of Heat-resisting Bituminous Pavements

While the results of efforts made in the U.K. and the U.S.A. to evolve suitable bituminous pavements to resist the effect of jet engines will be of immense value in indicating the solution to the problem encountered in hotter regions of India where high temperatures prevail during summer months, they cannot furnish the final solution. Applicability of the techniques under Indian conditions needs investigation.

The use of indigenous filler materials. e.g., mica dust, slate powder, asbestos flakes, have already shown promise in improving the heat-resisting properties of tar and bitumen. This aspect needs further investigation.

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APPENDIX 1

THE ELASTIC THEORY OF SOIL

1. Assumption and Limitations

The solution to the problem of stresses and displacements in a homogeneous deposit due to a concentrated load was first advanced by Boussenesq, and extended to soils by Biat and others. Assumptions made are :

(a) Soil deposit is an elastic, isotropic, homogeneous and semi-infinite body.

(b) Hook's law applies to soil deformation.

The investigations made by Hogg[3] in dealing with the conditions of equilibrium of a thin plate of finite extent symmetrically loaded on a flexible subgrade and by Burmister [4], who dealt with the theory of stresses and displacements in layered systems, are of special significance while considering the bearing capacity of superimposed layers. Hogg assumed that no frictional force occurs in the boundary surface between the slab and the subgrade although, later on, he gave an approximate idea of the effect of the frictional force in his calculations. Burmister took into consideration the effect of frictional force at the interface in the determination of the axial stresses at the interface and in the lower layer of the two-layer system. Also the following additional assumptions to those made by Boussenesq were made by him.

(a) The pavement over the subgrade is weightless and is infinite in extent in the horizontal direction only and the subgrade layer is infinite in extent both horizontally and vertically downwards.

(b) The two layers satisfy the "essential continuity conditions of stress and displacement across the interface" between two-layers. The condition of "continuity requires that the normal and shearing stresses as well as vertical and horizontal displacements must be equal in both the layers at the interface."

(c) "The two-layers are continuously in contact and act together as an elastic body of composite nature."

It will be evident from the assumption made by Burmister that a flexible pavement might, in all probability, satisfy reasonably the condition of continuity of stress and displacement. But Burmister maintains "that if a subgrade is continuously in contact with a concrete pavement and provides a reasonably uniform support and if the load is applied near the centre of a fairly large slab, the concrete pavement and the subgrade should act substantially in accordance with " his theory.

Although for a single concentrated load the displacement might be correct but in the case of a distributed load e.g., wheel load on a rigid pavement, the assumed proportionality of displacement at any given point away from the load is questionable[2].

In any case, Burmister's theory is inapplicable to the determination of stress and displacement in corner or edge loading of a concrete pavement, which, in many cases, is the ruling factor in the design of a concrete pavement.

The idealised subgrade and boundary conditions are only imperfectly satisfied by a natural soil or a subgrade. Also the general application of the theory to concrete pavement, has its limitations, yet the methods advanced by Hogg and Burmister reveal some of the fundamental relations between the factors which influence the load settlement characteristics of pavement and subgrade.

Calculation of stress (Boussenesq and Burmister)

Stress in a single homogeneous layer. For a single layor of homogeneous semi-infinite body, according to Boussenesq, if a concentrated load P is applied at the surface of the semi-infinite body, the vertical deformation w of the subgrade at a distance r from the point of application of the load is

$$w = \frac{P\,(1-\mu_s^2)}{\pi\; E_s \times r} \qquad \ldots \quad 1.1$$

Putting $C = \frac{E_s}{1-\mu_s^2}$, Equation 1.1

becomes $w = \frac{P}{\pi\, c.r.}$

According to the theory of elasticity, the settlement at the centre of a loaded rigid circular area (uniform settlement) resting on a homogeneous, elastic, isotropic solid of infinite depth is

$$W = \frac{P \times a \times \pi}{2} \times \frac{1-\mu_s^2}{E_s} \qquad \ldots \quad 1.2$$

where w represents the settlement of the loaded plate p is the intensity of loading in pounds per inch, a is the radius of the loaded area in inches, μ stands for Poissons ratio of subgrade and E_s is the modulus of elasticity of the subgrade in pounds per square inch.

Assuming that for a compacted soil μ is 0.5, Equation 1.2 becomes

$$w = 1.18\,\frac{p \times a}{E_s} \qquad \ldots \quad 1.2\,(a)$$

Similarly, the settlement at the centre of a uniformly loaded circular flexible area (equal reaction) is

$$w = \frac{2\;(1-\mu_s^2)\;p \times a}{E_s} \qquad \ldots \quad 1.3$$

If μ_s is assumed again as 0.5, the Equation 3 becomes

$$W = \frac{1.5\;p \times a}{E_s} \qquad \ldots \quad 1.3(a)$$

When a pavement of thickness h and modulus of elasticity E_p rests on a subgrade of semi-infinite depth and of modulus of elasticity E_s, the two layer system results.

Stress in a two-layer system

The theory of stresses and displacements in layered systems evolved by Burmister follows the pattern of Equations 2 and 3 for corresponding case of loading. In addition a settlement co-efficient F_w which is a multiplying (or correction) factor is introduced. Burmister's Equations are then, expressed as below:

(a) For a rigid bearing area

$$w = \frac{1.18\;p \times a}{E_s} \times F_w \qquad \ldots \quad (1.4)$$

(b) For a flexible bearing area

$$w = \frac{1.5\;p \times a}{E_s} \times F_w \qquad \ldots \quad (1.5)$$

The stress function "Fw" introduced by Burmister reveals that in the load-settlement relations, the controlling influences are exercised by two ratios, viz.,

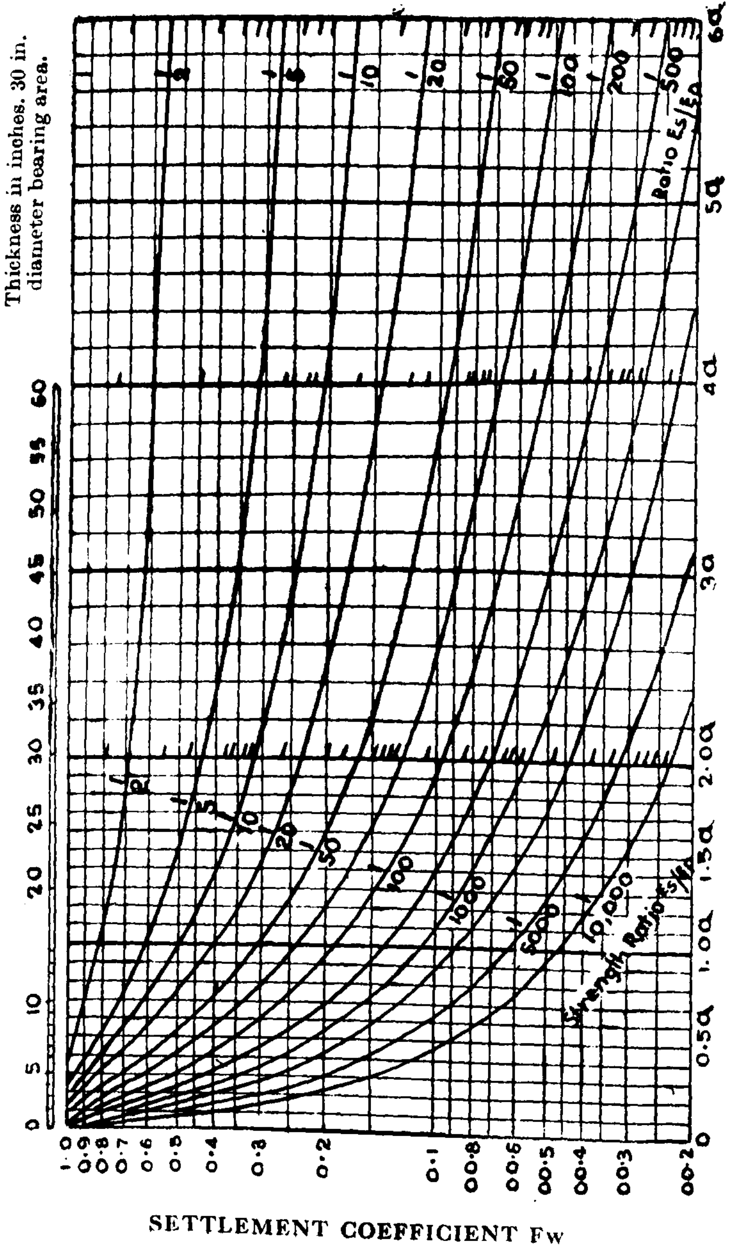
(a) the ratio a/h

(b) the ratio E_s/E_p.

Burmister's graph representing the variations in 'Fw' in respect of the ratios a/h for several values of E_s/E_p is reproduced on page 542.

It should be noted that when the thickness h is very great a/h approaches zero and the values of Fw become equal to the ratios of E_s/E_p.

If 1.5 pa, known as the Boussenesq factor remains constant, Table 1 shows the comparison between values of vertical displacements with the changes in the ratios $\frac{E_s}{E_p}$ for different values of $\frac{a}{h}$



Thickness h of the pavement expressed as a multiple of bearing area.

Fig. 1. Appendix 1. Graph of settlement coefficient Fw for a 2 layer system. $Fw = \frac{w \times Es}{1.5\ p.a.}$

TABLE 1, Appendix I

Variations in the values of vertical displacements with the variations in the strength ratios Ep/Es.

Ep/Es	Value of vertical displacement on the slab			Remarks
	$a/h=2$	$a/h=1$	$a/h=\frac{1}{2}$	
1	.97 w	.90 w	.80 w	'w' is the vertical displacement on the subgrade without the pavement
10	.74 w	.50 w	.31 w	
100	.44 w	.24 w	.13 w	
1000	.28 w	.11 w	.054 w	

It is seen from Table 1 that when the pavement has the same modulus of elasticity as that of a subgrade there is very little reduction in the vertical deformation under the load. Increase of the thickness of the pavement 4 times when $E_p=E_s$, results in the reduction of the vertical displacement by only (.97 w – 80 w) 0.17. On the other hand, the effect of increasing the thickness of the pavement in reducing vertical displacement is more pronounced as the modulus ratio E_p/E_s increases.

The method advanced by Burmister is utilized by U. S. Department of Navy (Bureau of Yards and Docks) in determining the required thickness of flexible pavements[24]. On the other hand, Robert Ruckli, [25] is of the opinion that because they have practically no tensile strength, flexible pavements cannot be analysed by applying Burmister's theory to cases where the ratio Ep/Es is small [24].

The US Navy Method of designing flexible pavement based on Burmister theory is to determine the values of E_s and E_p by plate-bearing tests [24]. E_s is measured in the field by a plate-bearing test utilising a 30-in. diameter plate overlaid with several plates of smaller diameters giving the effect of a rigid plate. The value of E_s is then derived from Equation 1.2 (a). E_p is measured by a plate-bearing test in a similar manner on a trial section of pavement and is determined from the equation 1.4.

APPENDIX 2

THEORY OF ELASTIC FOUNDATION (WESTERGAARD)

Assumptions

The following assumptions are made by Westergaard.

(a) the slab acts as a homogeneous, isotropic and elastic solid in equilibritm.

(b) the subgrade reaction acts vertically and is proportional to the deflection of the slab.

(c) the value of k is the same at all points within the area under load and is independent of the deflection. Minor variations in the value of k can be of no great consequence.

Relation between elastic semi-infinite body and elastic foundation

The value of k is obtained by applying a load to a stiff circular plate of standard diameter and measuring the vertical displacement. If the load is assumed to be uniformly distributed over the plate, the modulus of subgrade reaction k can be computed by the observed deformation w. The value of k is a measure of the stiffness of the subgrade and is expressed by the equation

$$k = \frac{p}{w} \quad \ldots \; 2.1$$

By applying Boussenesq's well known formula for the settlement of a stiff foundation we get,

$$c = p/w \times \frac{\pi \times a}{2} = k.\frac{\pi.a}{2}$$

But the co-efficient of subgrade reaction k is a function of the size of the bearing plate. However the value of k can be converted into the basic subgrade modulus E_s in a layered system by the following Equation 4.

$$k = \frac{p}{w} = \frac{E_s}{1.5\ {}_aF_w} \text{ or } E_s = 1.5 \times a \times k \times Fw \ldots \; 2.2$$

Similarly for rigid pavement $E_s = 1.18 \times a \times k \times Fw$ 2.3

The application of this formula will give slightly different values of k for different thickness of pavement. This is confirmed by the investigation carried out by Beijer ([2]).

Westergaard's principal formulae are given below.

In computing the stresses in a rigid pavement, Westergaard took into consideration the dynamic effect of the wheel load[11] whereas the calculations based on the elastic theory given in Appendix 1 are restricted to static wheel loads.

In a concrete slab the corner loading generally produces the maximum stress and the interior loading the least stress[78].

WESTERGAARD'S FORMULAE

Using a value of Poisson's ratio $\mu = 0.15$ the critical stress in a concrete pavement slab as induced by traffic loads according to Westergaard is expressed by the following formulae.

Original

Interior loading $S_i = \dfrac{0.3162P}{h^2}\left[4\log_{10}\left(\dfrac{l}{b}\right) + 1.069\right]$ 2.4

Edge loading $S_e = \dfrac{0.572P}{h^2}\left[4\log_{10}\left(\dfrac{l}{b}\right) + 0.359\right]$ 2.5

Corner loading $S_c = \dfrac{3P}{h^2}\left[1 - \left(\dfrac{a\sqrt{2}}{l}\right)^{0.6}\right]$ ... 2.6

in which S_i, S_e, S_c = maximum stress in pounds per square inch for interior, edge and corner loading respectively.

b = radius of resisting section $= \sqrt{1.6\,a^2 + h^2} - 0.675\,h$

Revised

Interior loading $S_i = \dfrac{0.316P}{h^2}\left[4\log_{10}\left(\dfrac{l}{b}\right) + 0.633\right]$ ... 2.7

Corner loading (Sprangler)

$$S_c = \frac{3P}{h^2}\left[1 - \left(\frac{a}{l}\right)^{0.6}\right] \quad \ldots \; 2.8$$

Corner loading (Teller and Sutterland)

$$S_c = \frac{3P}{h^2}\left[1 - \left(\frac{a}{l}\right)^{1.2}\right] \quad \ldots \; 2.9$$

Temperature Effect

$$\text{Centre warping } S_t = \frac{E \times e \times t}{2} \times \frac{K_{wt} + \mu K_{w2}}{1-\mu^2} \quad \ldots 2.10$$

$$\text{Edge warping } S_t = \frac{K_w \times e \times E \times t}{2} \quad \ldots 2.11$$

$$\text{Corner warping } S_t = \frac{E \times e \times t}{3(1-\mu)} \sqrt{\frac{a}{l}} \quad \ldots 2.12$$

S_t is the warping stress and K_w, K_{w1}, K_{w2}, are obtained from Figure 14.

Some authorities consider that formula 2.7 underestimates the stress condition and that it is reasonable to use a formula which will over-estimate.

APPENDIX 3

REVIEW OF SOME TESTS

De Havilland airfield, Hatfield.

30-inch diameter plate bearing tests[1] were carried out on the prepared subgrade and on the following experimental concrete pavements.

(i) Six panels of 12-in. concrete on 3-in. concrete.

(ii) Sandwich slabs consisting of 7-in. concrete over which was laid a 6-inch concrete slab with building paper as a separating medium.

(iii) Nine panels of weak concrete on a 3-in. base of concrete 12 : 1.

(iv) Six Panels of 8-in concrete on 3-in. base of concrete 12 : 1.

(v) Six panels of 8-in concrete laid on a campacted sub-base.

In the sandwich construction the points of the upper and the lower layers were staggered so that the corner of a slab in the upper layer rested on the centre of the lower layer. The stress in concrete computed from the observations of the 30-in. diameter plate bearing test showed excesive stress in concrete, viz.

(a) at the centre of the 6-in. top slab, the maximum stress was 877.5 lb per sq in.

(b) near the edge of the 6-in. top slab, the maximum stress was 1523 lb per sq in.

The stresses in concrete in other experimental slabs were within 500 lb per sq. in. The average value of modulus of sub-grade reaction k was 474 and the average value of the modulus of elasticity of concrete in the pavement varied from 3,900,000 lb. per sq. inch to 5,800,000 lb. per sq. in.

Before the construction of the airfield at Hatfield, the results of investigation of k values of original subgrade, compacted subgrade and a lean-concrete sub-base were taken into account. These tests were carried out by plate-bearing tests and vaiations in the values of k as investigted by Snow[1] are reproduced below :—

		16 per sq. in.
(a)	Modulus of subgrade reaction - k Original soil with 30-in. plate	300
(b)	Formation prepared by removing 18 inches of top soil and replacing by selected fill material in layers rolled and consolidated with sheepsfoot rollers and heavy rubber-tyred vehicles to the total thickness of 12 inches (average of 11 tests with a 30-in. plate)	474
(c)	for the same subgrade as in (b) but exposed to heavy rains necessitating replacement of slurry and replacement by hard-core blended with ash, with 30 in. plate.	382
(d)	a 3-in. lean concrete 12 : 1 after 5 days curing, with 18-in. plate.	1770

Data which would be helpful in checking the observed results with those obtained by applying the theory of layered system are not available. In fact, during the discussion of this Paper it was pointed out that the works of Burmister and Fox should have been studied and the increase in the bearing established power of the subgrade with the relative elasticity of the layers. However, the results published show the extent of variations in the k values of the subgrade at Hatfield due to variation in base course.

Trials Carried out by the U. S. Corps of Engineers

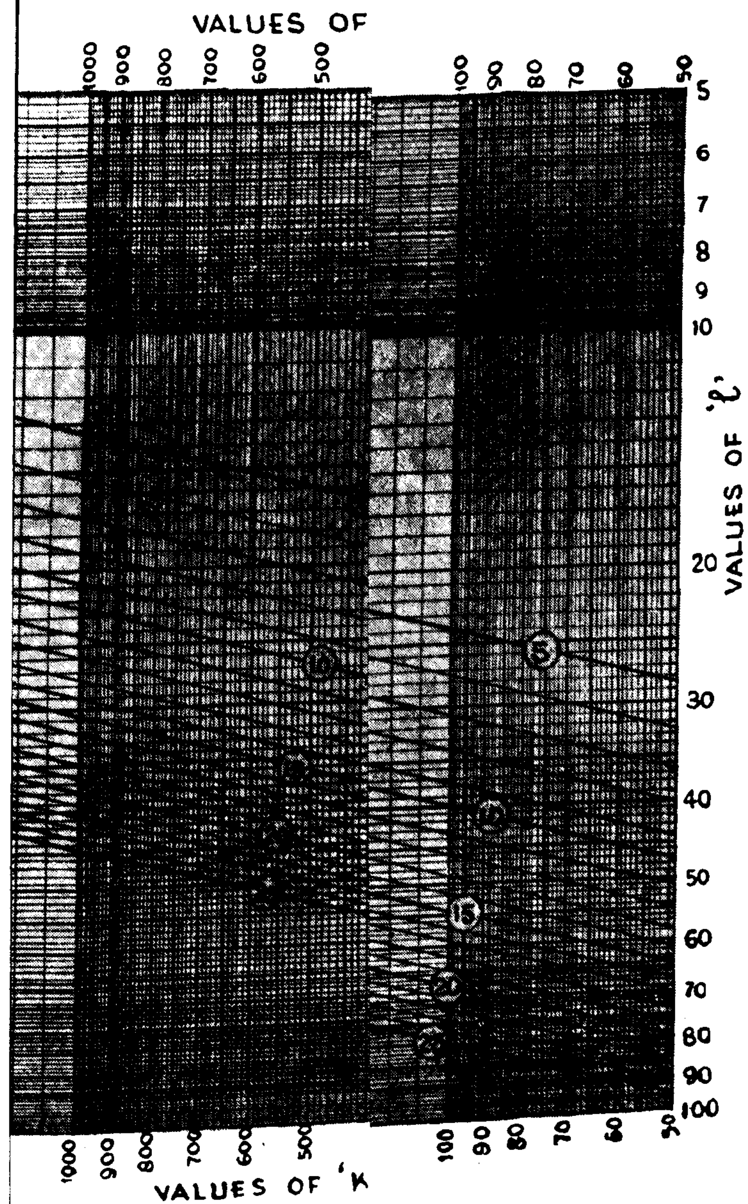
Full scale tests[26] at three airfields were conducted by the U. S. Corps of Engineers with a 60,000–lb load on dual wheels of a B–29 aircraft configuration on a concrete pavement 6-in. thick with flexible types of overlays 3 to 12 inches thick and with rigid type overlays 6 to 9 inches thick.

The test section at the MacDill Field airfield had concrete overlays 6 inches and 8 inches thick on the existing pavement with a 1/2 in. sand-asphalt cushion. Both the test sections developed cracking at joint intersections. The cracking was greater in the overlays than on the existing pavement and was considered excessive for a satisfactory surface.

On the other hand, a 9-inch flexible pavement consisting of 6-in. lime rock base with a 3-in. asphaltic concrete surface provided the best reinforcement for the 6-in existing concrete pavement.

At the Maxwell test field, flexible pavement overlays 12 and 9 inch thick consisting of a 9 or 6 inch penetration macadam base and a 3-in. asphaltic concrete surface, provided adequate reinforcement to prevent crack development in the existing pavement. Also a cement concrete overlay used for maneouvre area withstood uncontrolled maneouvre traffic with no significant cracking. From other tests it was found that a bituminous separation course, even as thin as a prime coat, was conducive to cracking of concrete overlays.

PLATE I



stiffness in inches;

ES OF 'h', 'k' AND 'l'

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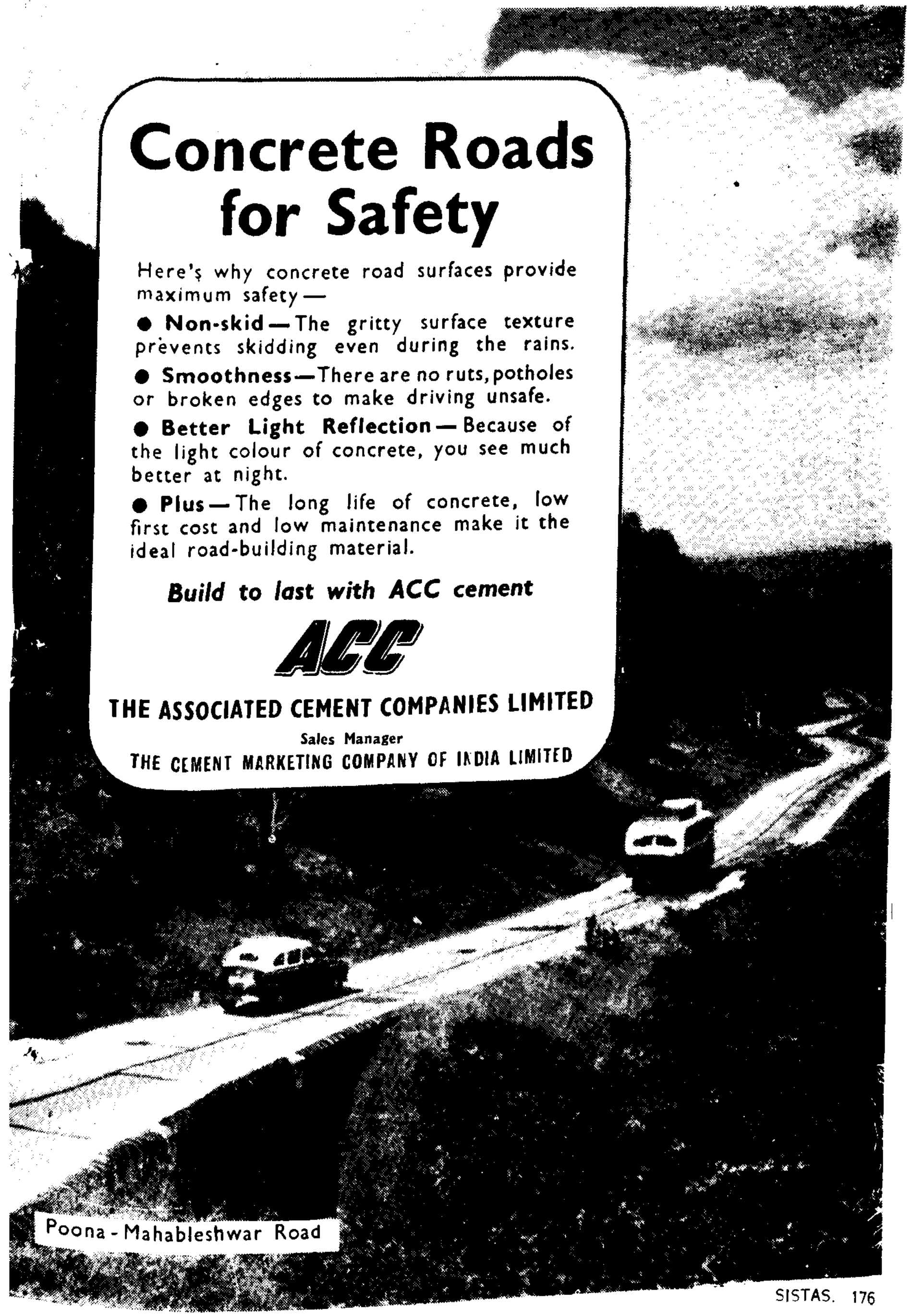
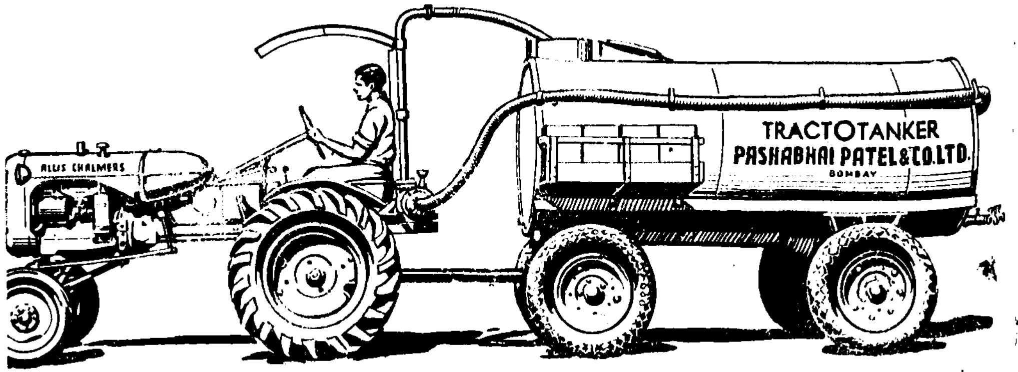
n.;

36 for E=20,00,000, by 1.1074 for

.01 for μ=0.25


Concrete Roads for Safety
Here's why concrete road surfaces provide maximum safety—
● Non-skid—The gritty surface texture prevents skidding even during the rains.
● Smoothness—There are no ruts, potholes or broken edges to make driving unsafe.
● Better Light Reflection—Because of the light colour of concrete, you see much better at night.
● Plus—The long life of concrete, low first cost and low maintenance make it the ideal road-building material.
Build to last with ACC cement
ACC
THE ASSOCIATED CEMENT COMPANIES LIMITED
Sales Manager
THE CEMENT MARKETING COMPANY OF INDIA LIMITED
Poona - Mahableshwar Road

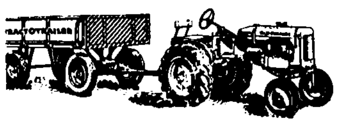
TRACTOTANKER



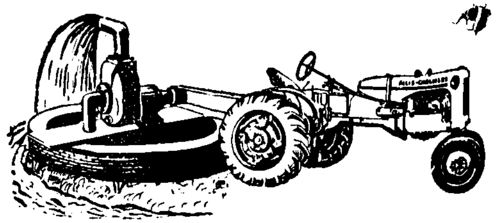
Originally designed to fight the water famine in India, this versatile TractOtanker unit has now become the road engineer's godsent gift.

The *Allis Chalmers* TractOtanker has a pump which fills the 700-gallon tank in three to five minutes only. The entire unit can turn back even on the narrowest road.

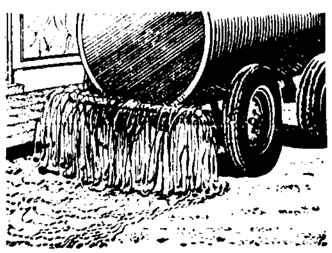
This unit has become very popular among road engineers for sprinkling water on earthfills and road-construction jobs; for carrying water and oil economically over long distances, watering road-side trees, and for many other uses. Ask us for full details of the many uses of this road engineer's most modern tool.



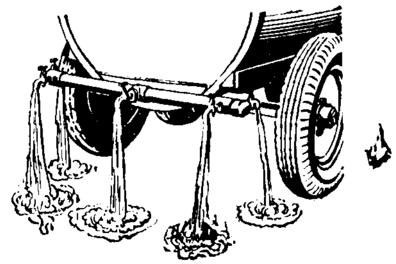
TRACTOTRAILER



IRRIGATION

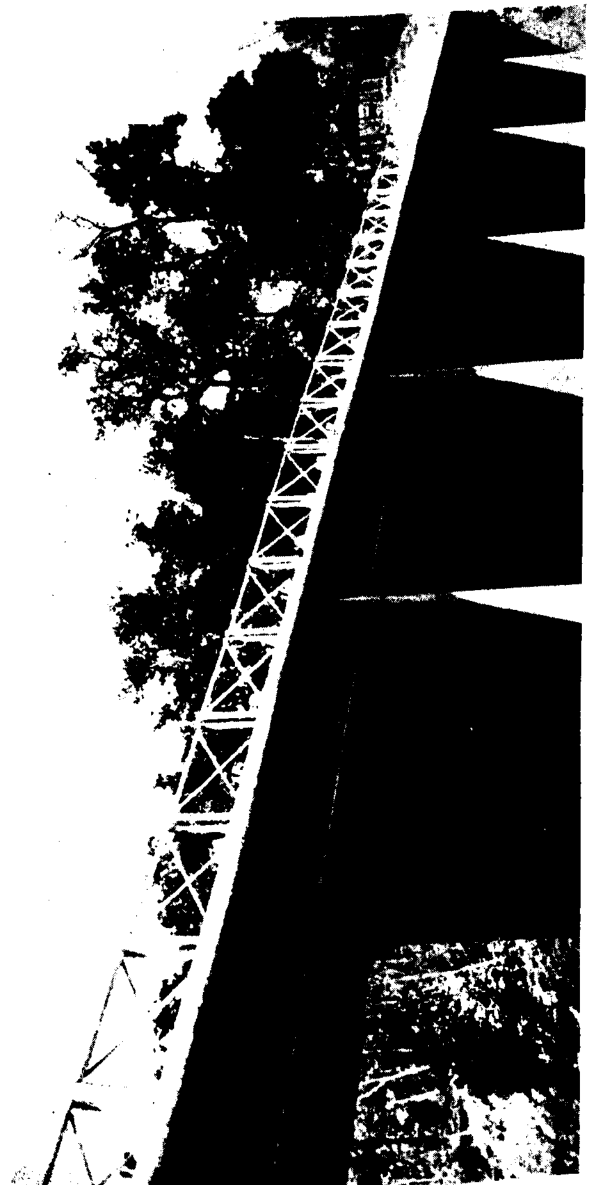


MUNICIPAL SPRINKLER



WATER TAPS

PASHABHAI PATEL & Co., LTD. CONSTRUCTION HOUSE BALLARD ESTATE BOMBAY



TUMNAR NALLAH BRIDGE—A reinforced concrete submersible bridge 352 feet long and is made up of two units of rigid frame. One unit consists of two spans 32 ft. each and two spans 40 feet each with a 12 feet cantilever on one side. The other unit consists of two spans 42 feet each, two spans 50 feet each with a 12 feet cantilever on one side. The decking consists of solid slab near supports which is troughed in the middle portion of the span. The wall type piers are of uniform cross section and are monolithically connected to the solid slab. The piers rest on mass concrete foundations taken down to rock by open excavation.

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