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The Indian Roads congress

1957-58

The Indian Roads Congress as a body does not hold itself responsible for statements made, or for opinions expressed, in the Papers, Research and Information Sections or discussions in its Proceedings, etc.

Dr. Rajendra Prasad

President of the Republic of India

is pleased to be the

Patron

of the

Indian Roads Congress, New Delhi.

By Command of the President of the Republic of India

Military Secretary's Office.
The 5th March 1951.

B. Chatterjee, Colonel.
Military Secretary to the President

INDIAN ROADS CONGRESS

ESTABLISHED : 1934 REGISTERED : 1937

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A representative of Central Standards Office for Railways
A representative of the Poona Irrigation Research Station
A representative each from Lucknow, Madras, Karnal, Bihar & Hyderabad Research Stations
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Consulting Engineer (Road Development)
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The Consulting Engineer to the Govt. of India (Road Development) is an ex-officio Member of all Committees.

INDIAN ROADS CONGRESS

NOTICES AND ANNOUNCEMENTS

I. CONTRIBUTION OF PAPERS FOR THE TWENTY-SECOND SESSION OF THE INDIAN ROADS CONGRESS

The next General Session of the Indian Roads Congress will be held at New Delhi from the 4th to the 9th January, 1958. Members are requested to contribute Papers on any subject connected with Highway Engineering for discussion at the Session.

2. On the occasion of the next Indian Roads Congress Session a Seminar on **Low cost roads** will also be organized by ECAFE. As the Seminar on low cost roads will immediately follow the Indian Roads Congress Session, it is suggested that good Papers on low cost roads incorporating the experience gained by various engineers in India should be contributed to the Indian Roads Congress so that the foreign delegates attending the Seminar could also participate in the discussion on these Papers.

3. **Procedure:** Members who intend to contribute Papers on any subject connected with Highway Engineering are requested to intimate the fact to the Secretary, Indian Roads Congress, and to send a short synopsis of their proposed Papers as early as possible.

On receipt of acceptance of the subject matter of the Paper, the Member concerned should send his contribution to the Secretary, Indian Roads Congress, complete in all respects and with original drawings, photographs, etc., as and when ready. At least **three** copies of the typescript matter should be sent, typed (double space) on one side of good stout paper of foolscap size.

4. **The complete Paper, if intended to be read at the next Session, should be submitted before 15-11-57.**

From past experience it is observed that most of the Members send in their contributions at the last moment, when the Congress Office is busy on work connected with the Session. As it takes a long time to get Papers printed in the Journal and as it is desirable that Members should have time to study Papers well in advance of the date of the Session, it is hoped that

Members will make an effort to send their contributions as early as possible.

Members may please note that Papers received late in the year are likely to be carried over to the next year.

5. The younger engineers are particularly requested to contribute Papers which, if required, will be suitably edited before publication.

6. **MEDALS:** The following Medals are available for award:

(*i*) **The Mitchell Medal:**

For exceptionally good Papers contributed to the Indian Roads Congress in any year. This has been instituted to perpetuate Sir Kenneth Mitchell's connection with the Congress.

(*ii*) **The Indian Roads Congress Medal:**

For Papers considered by the Council of the Indian Roads Congress to be of a decidedly more than average merit.

7. The Council have reserved the right to award more than one Medal in any year or to withhold the award of a Medal in any year without assigning any reason. All Members are eligible for the award of the Medals.

8. Rules and Regulations for the contribution of Technical Papers and the award of Medals are published on pages 9 to 14.

II. RULES AND REGULATIONS FOR THE CONTRIBUTION OF TECHNICAL PAPERS AND THE AWARD OF MEDALS

1. The following Medals are available for award:

(*i*) **The Mitchell Medal:**

This is a bronze medal to be awarded annually for exceptionally good Papers contributed to the Indian Roads Congress in any year. This has been instituted to perpetuate Sir Kenneth Mitchell's connection with the Indian Roads Congress.

(*ii*) **The Indian Roads Congress Medal:**

This is also a bronze medal to be awarded annually for Papers considered by the Council of the Indian Roads Congress to be of a decidedly more than average merit.

2. PROCEDURE FOR CONTRIBUTION OF TECHNICAL PAPERS

2.1. Our procedure in the past had been to invite Members to contribute Papers a few months before the holding of the General Sessions of the Indian Roads Congress. On receipt, the accepted Papers were circulated in advance to Members for prior study a month or so before the date fixed for discussion. This Procedure resulted in the receipt of a large number of Papers all at one time and editing and printing had often to be done very hurriedly. Also Members often found that one month did not give them sufficient time to study all the Papers before the discussion at the Congress Session.

2.2. It has, therefore, been decided by the Council that Members should be invited, to contribute technical Papers throughout the year, as and when they are ready, and that the accepted Papers should be published in the Journals of the Society which, it is hoped, will be issued quarterly in future.

2.3. The Council considers that this will give Members more time to study the Papers and will also facilitate editing and printing. A record of the discussion and the comments received by correspondence will be published with the Author's replies in the Journal issued after the General Session.

2.4. It will be realized that it will only be possible to give effect to this new scheme if Members co-operate. The Council makes this special appeal to members to contribute Papers for discussion throughout the year at their convenience.

3. RULES FOR THE CONTRIBUTION OF TECHNICAL PAPERS

3.1. The Standing Rules for guidance when contributing Papers, as now slightly amended, are reprinted below. It is requested that Members read these rules and observe them when submitting Papers to the Congress.

3.2. Members intending to contribute papers to the Indian Roads Congress on any subject connected with Highway Engineering are requested to send their contributions to the Secretary as and when these are ready without waiting for a special invitation.

3.3. Final acceptance of the Papers rests with the Council. In order, however, to avoid disappointment through the Council having to reject Papers owing to an excessive number being written on any one subject or for any other reason, it is requested that any Member intending to write a Paper should intimate to the Secretary the subject on which he wishes to write and enclose a short synopsis of the proposed Paper. He will then be informed whether his offer can be accepted.

3.4. The Council reserve the right to publish any Paper in the form of an abstract. When a Paper is published in abstract form the manuscript copy of the Paper as sent by the Author will be added to the Indian Roads Congress Library and made available for inspection by interested Members.

3.5. The Council have decided that Papers from non-Members who are experts or who have specialized knowledge will be welcome and acceptance of the Paper will carry honorary membership for the Author for the year provided there are good reasons that the contributor cannot join the Congress as an ordinary member.

3.6. The Council have expressed the opinion that Members, when choosing subjects for contributing Papers, should bear in mind the fact that there is often a good deal more to be learnt from failures than from success.

4. SCRIPT

4.1. The following instructions are for the guidance of Authors of Papers ;

a. **All contributions should be drafted in the third person.**

b. Papers should be typed on one side of good stout paper of foolscap size, double spacing being used and an ample margin (not less than 2 inches) should be left on the left hand side. **At least three copies of the typescript matter should be sent.**

c. The Author's full name, designation, academic qualifications and correct postal address should be given below the title of the Paper.

5. ILLUSTRATIONS

5.1. Illustrations fall into two classes, *viz.*,

a. Line drawings.

b. Half-tones.

5.2. *Line drawings* should be made in black Indian ink on superior white drawing paper or tracing cloth. Blue prints are of no use for reproduction and black line prints are also unsatisfactory.

Line drawings are generally classified into two categories:

a. Diagrams which go into the body of the Paper.

b. Plates which are generally on separate large size sheets.

5.3. *Diagrams*: The matter for the Journal is printed in 10 pt. size type (nearly $\frac{1}{16}$ in. in height); therefore, the written matter in diagrams should be so made that the size of the letter when reduced for reproduction should be about 1/16 in. The printed area of a page measures $6\frac{3}{4}$ in. deep by $4\frac{1}{4}$ in. wide. Therefore, the ultimate size of such drawings will not generally be more than 5 in. $\times$ 4 in. Thus, if a diagram is to be reduced to one quarter of the size submitted by the Author, the lettering on it should not be less than $\frac{1}{4}$ inch in height.

5.4. *Plates*: Plates should preferably be prepared in widths that are multiples of $7\frac{1}{2}$ in. The size of letters used in the plate should be so chosen that after reduction no letter would be less than 1/16 in. in size. Thus, if the width is 15 in. the minimum size of the letter should be $\frac{1}{8}$ in. The thickness of the letter should also stand corresponding reduction. The title should be in the right hand bottom corner in letters of such size that when reduced the size will be at least $\frac{1}{8}$ in. The name of the Author may be printed, if desired, at the bottom right hand side.

5.5. Scale should be drawn in the plate below the title to admit of reduction of drawings without altering the correct relation of the scale to the drawing. Mere mention of the scale thus "Scale 10 ft=1 inch" should be avoided as this would be incorrect when size of the plate was reduced photographically.

5.6. Coloured inks should not be used. When it is desired to distinguish lines, dotted or chain dotted lines should be used instead of colours.

5.7.1. In drawing diagrams or graphs, the following points deserve the consideration of Authors.

5.7.2. From the standpoint of pleasing appearance, a rectangular graph or drawing with proportions between 3 by 5 and 3 by 4 is to be preferred to a square one.

5.7.3. The appearance and effectiveness of a graph depend in a large measure on the relative thickness of lines used in its component parts. The thickest line should be used for the principal curve. If several curves are presented on the same graph, the line width used for curves should be less than that used when a single curve is presented. Co-ordinate rulings should be the narrowest in thickness. Principal reference lines such as the axes should be wider than other rulings but narrower than curves. For the size of reduction that is usually adopted, for the I. R. C. Journals it is considered that the thickest line *when finally reduced* should not be more than 2½ points, *i.e.*, 0·035 inch in width.

5.8. Finished illustrations should not be folded for purposes of despatch. When mailed flat, they should be stiffened by using card boards or rolled with the drawing outside to prevent cracking of the Indian ink lines and securely packed.

5.9. "*Half-tones*" are ordinary photographs and it is necessary to see that prints are clear, slightly over printed and preferably glazed. Photographs which are dim or out of focus do not come out distinctly in reproduction and make a bad "half-tone". Negatives should accompany the photographs whenever possible, and will, if desired, be returned when done with. Captions should be written on the back of prints in soft pencil.

6. BRIDGE DETAILS

6.1. Members contributing Papers on any bridge work actually carried out are requested to give *inter alia* the following details.

a. A site plan showing the road and river for two miles on either side of the bridge site;

b. A dimensioned cross-section of the roadway;

c. The loading on which the design is based;

d. The cost per square foot of the elevation area comprised by the formation level and the bottom of the foundations; and

e. The ratio of the total cost of the substructure to the cost of those parts of the superstructure as are subject to variation with variation of span.

7. UNITS

7.1. Members are particularly requested to use the standard units of weight, measure, and cost given below:

STANDARD UNITS OF WEIGHT AND MEASURE

	Description of works	Units to be adopted
1.	Road material such as metal gravel or moorum, sand	100 cu. ft.
2.	Binder in surface dressing and grout	lb per 100 sq. ft.
3.	Binder in premix, carpets, etc.	lb per cu. ft. of aggregate.
4.	Cement	By weight and not by volume
5.	Water-Cement ratio	gallons of water per cu. ft. of cement (assumed as 90 lb)
6.	Cost and Rates	In Indian Standard currency (Rupees, etc.)

8. FORMULAE

8.1. Mathematical formulae should be drawn carefully by hand and not set out with the typewriter.

9. ABBREVIATIONS

9.1. The use of symbols should be avoided. For example 6′-2″ × 3′-3″ should be written 6 feet 2 inches by 3 feet 3 inches. Similarly 225° C. should be written as 225 degrees centigrade, and so on.

10. ITALICS AND EMPHASIS

10.1. Only those words and phrases which are to be printed in italics should be underlined.

11. PARAGRAPHING

11.1. When divisions and sub-divisions are designated by numbers and letters, the following symbols may be used in the arrangements indicated.

I.
II.

1.
2.

a.
b.

(i)
(ii)

12. ACKNOWLEDGMENTS

12.1. Sources of quotations appearing in the Papers should be stated and acknowledgment should be made for all information culled from books, periodicals, and proceedings of sister Societies, etc.

III. PROCEDURE REGARDING THE DISCUSSION OF PAPERS AT THE ANNUAL SESSIONS OF THE INDIAN ROADS CONGRESS

1. The order in which Papers will be taken up for discussion will be decided by the Council and intimated to members attending the Session.

2. The Chairman will introduce the Author and will call upon him to present his Paper for discussion. The Paper will be taken as read. Introductory remarks shall not ordinarily take more than 10 minutes and, unless the Author had something new to say which came to his notice since the submission of his Paper, the introduction should be limited to 250 words. After the introduction, the Paper will be open to discussion.

3. To facilitate the conduct of business and the fixation of time for various sittings and individual Papers, those desirous of raising questions or offering comments at the meeting should inform the Secretary well in advance (*at least 24 hours before the meeting*) about the Papers on which they would like to speak and the time they would take. Those who have not given the required intimation to the Secretary may, at the discretion of the Chairman be allowed to take part in the discussion if time permits and after those who have intimated their intention to speak in advance have spoken.

4. A copy of or the nature of the comments which a member intends to make should be sent in advance to the Author of the Paper and a copy endorsed to the Secretary.

5. If the comments are likely to take more than ten minutes or if they involve lengthy mathematical matter or diagrams, it would be desirable to send a copy of these to the Secretary at least 2 weeks before the date of the meeting so as to enable him to have them printed and circulated to all members attending the meeting before the discussion on the Paper takes place. These comments may, at the discretion of the Chairman, be taken as read and the member making them will thus only be required to explain generally the nature of his comments. The author will, if possible, reply at the meeting as if the comments had been made orally.

6. Members who do not want to speak but want clarification of any issues arising out of the discussion or the introductory remarks of the Author should write out their queries on

"Comment Slips" which will be provided for the purpose while the discussions are in progress. Volunteers will be deputed to distribute and collect these slips at frequent intervals. The slips will be sorted out by the Secretary and at the end of the talk by the Speaker or Speakers, the Chairman will read out the queries and the Speaker or the Author will give a reply.

7. Members who are unable to attend the Session and who desire to comment on any Paper should send one copy of a note of their comments to the Author of the Paper and two typescript copies to the Secretary. A gist of the comments together with the Author's reply thereto will appear in the Proceedings.

8. After all members wishing to do so have spoken on a Paper, or when the Chairman considers that because of lack of time the discussion must be brought to a close, the Author will be called upon to reply.

9. If the author of a Paper is unable to be present, he should depute another member to introduce his Paper and to reply to the discussion and should inform the Secretary accordingly.

IV. LIBRARY

1. Information and books on loan from the libraries of the Consulting Engineer (Road Development) and the Indian Roads Congress are available to **Members** of the Indian Roads Congress and can, on application, be had of the Secretary, Indian Roads Congress, New Delhi under the following arrangements :

(a) **For Members of the Indian Roads Congress who are in the service of the Central or State Governments.**

Specifically named books or literature in response to enquiries on technical subjects will be sent to officers of the rank of Executive Engineer and above. Others should apply through their Executive Engineers or officers of equivalent rank.

(b) **For Members of the Indian Roads Congress who are not in the service of the Central or State Governments.**

Applications for specifically named books as well as enquiries for literature on technical subjects should clearly state the member's roll number, his correct postal address, and the title and author of the book.

2. The library is intended primarily for reference. It is the source of information from which the Secretary, Indian Roads Congress, is enabled to answer enquiries addressed to him.

3. **Books are loaned for reference only and not for continuous use.** The library cannot be expected to relieve officers of the necessity of purchasing books required constantly or for long periods of time.

4. Ordinarily not more than two books will be issued at a time, but in special cases a larger number may be issued at the discretion of the Secretary.

5. The following rules will be strictly enforced :

(a) Books are issued for one month in the first instance. If a book is required for a longer period, an intimation should be sent to the Secretary, Indian Roads Congress for renewal before the due date of return. The book may then be re-issued for another month at a time at the discretion of the Secretary but in no case will any book be issued for a total period exceeding three months.

(b) If a member does not return library books when the period of loan has expired or in any case within three months of the date of issue and thus deprives other members of their use, he will be liable for a fine at the rate of one anna a day for every day of retention upto three months and two annas a day thereafter upto six months. The Secretary will also have no power to issue any more books to the member until all the books already issued are returned. If all books are not returned within six months, twice the cost of the books will be debited to the borrower in addition to the fine referred to above.

(c) Current issues of Journals cannot be issued to members outside the Consulting Engineer (Road Development's) Office, but back issues of certain Journals may be sent on loan for a limited period.

(d) Books are issued strictly subject to the condition, in all cases, that they will be returned promptly if actually required and called for by the Secretary, Indian Roads Congress. Retention of a book after it had been recalled will render the member liable to the same penalties as if he had retained a book beyond the period of original loan.

(e) Borrowers are requested to take every care of books while in their possession, and particularly not to mark, underline, or note on the pages.

(f) If, during the period of issue to a borrower, any book is lost or so damaged as to be unserviceable for future use, the Secretary shall recover from the borrower a sum equal to the cost of obtaining another copy of the book.

6. **Instructions**:

(a) No acknowledgment is required other than the postal acknowledgment card.

(b) While returning books, care should be taken to get them adequately packed to prevent damage in transit. They should be sent "**registered** and **acknowledgment due.**"

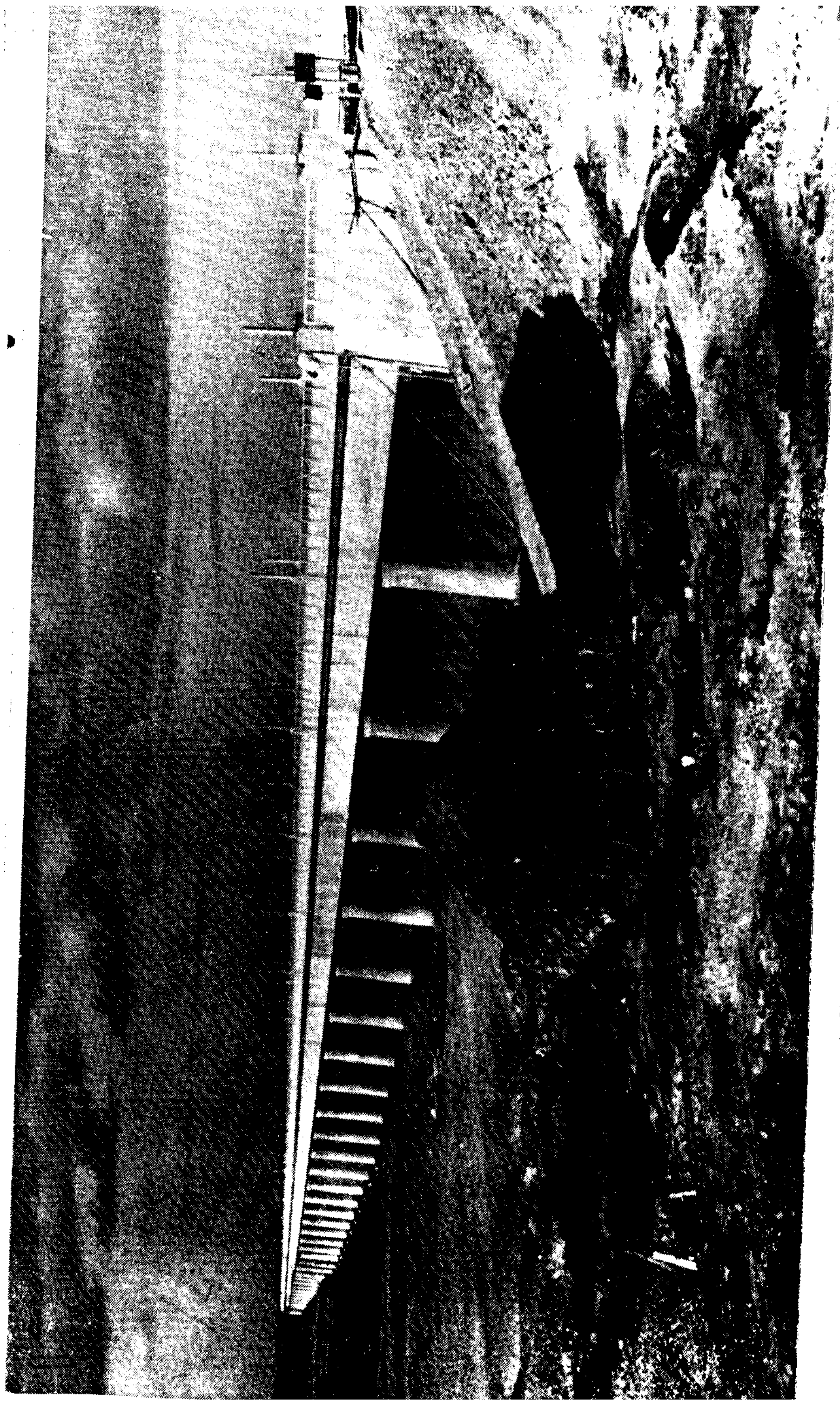
7. **General**:

(a) Rare or specially valuable books and certain books of reference will not be sent out of the library.

(b) Any book may, however, be consulted in the library during office hours.

(c) Members who retain books for longer than a month after they have been called back or for longer than six months from the date of original issue and thus selfishly deprive other members of the use of the books, will be fined and/or charged, subject to the penalties stated in these rules and deprived of all the privileges of membership until the dues are paid.

NOTE: All Members of the Indian Roads Congress, not in arrears of subscription for more than 90 days, are entitled to borrow books.



A VIEW OF THE COMPLETED BRIDGE OVER PENNAR

Paper No. 199

"CONSTRUCTION OF A BRIDGE ACROSS PENNAR RIVER AT NELLORE"

By

J. V. PRASAD*, B.E.

CONTENTS

SYNOPSIS

The Paper describes the construction of the Pennar Bridge on National Highway No. 5 in Andhra State. The bridge consists of 27 spans of 71 ft each and 10 spans of 61 ft each R.C. T-beams simply supported. The foundations are partly open and partly on wells. The piers and abutments are of mass concrete. The roadway is 22 ft wide with 5 ft cantilever footpaths.

The general features of the bridge are described and the details of construction which was done departmentally are given, including the various difficulties encountered during execution of the work. Details of cost are also included.

The Paper also describes the load test of the superstructure.

* Assistant Engineer (Highways), Madanapalli, Chittoor District.

1. INTRODUCTION

1.1. The Madras-Calcutta Road, the National Highway No. 5, crosses the river Pennar at mile 109/1 to 4 from Madras. This is one of the major rivers on this route. The river Pennar rises in the Nandidurg hills in the Mysore State and is 330 miles long passing through the districts of Ananthapur, Cuddapah and Nellore of the Andhra State. The river joins the Bay of Bengal about 15 miles from the bridge site.

1.2. There is a P.W.D. anicut across the river and the traffic on this road was crossing the river by the anicut. The crossing was very rough with narrow windings, steep and awkward approaches. There would be freshes in the river for about 60 days in a year and during that period there would be complete cessation of traffic on this National Highway. The traffic on the Madras–Calcutta road is always heavy and the river bisects the Nellore district into two portions. The want of a bridge was, therefore, keenly felt by the public.

1.3. The construction of the bridge was commenced in December 1949 and the bridge was opened to traffic in January 1955.

2. LOCATION

The location of the bridge is 300 ft downstream of the centre line of the existing railway bridge. This site was the best as it affords a well defined narrow section between the banks compared to the wide and shallow sections both upstream and further downstream side of the present bridge site as seen in the Key Map and the Site Plan, Figs. 1 & 2. Further, laterite rock was available at the surface for about quarter the length of the bed which has reduced the cost of the bridge considerably.

3. CATCHMENT AND DISCHARGE

The catchment area of the river and its tributaries is about 20,000 square miles. The maximum discharge provided is 663,548 cusecs which is the discharge over the anicut observed in December 1946. The maximum flood level at the railway bridge is 58.00 above M.S.L. and the same was adopted for the road bridge.

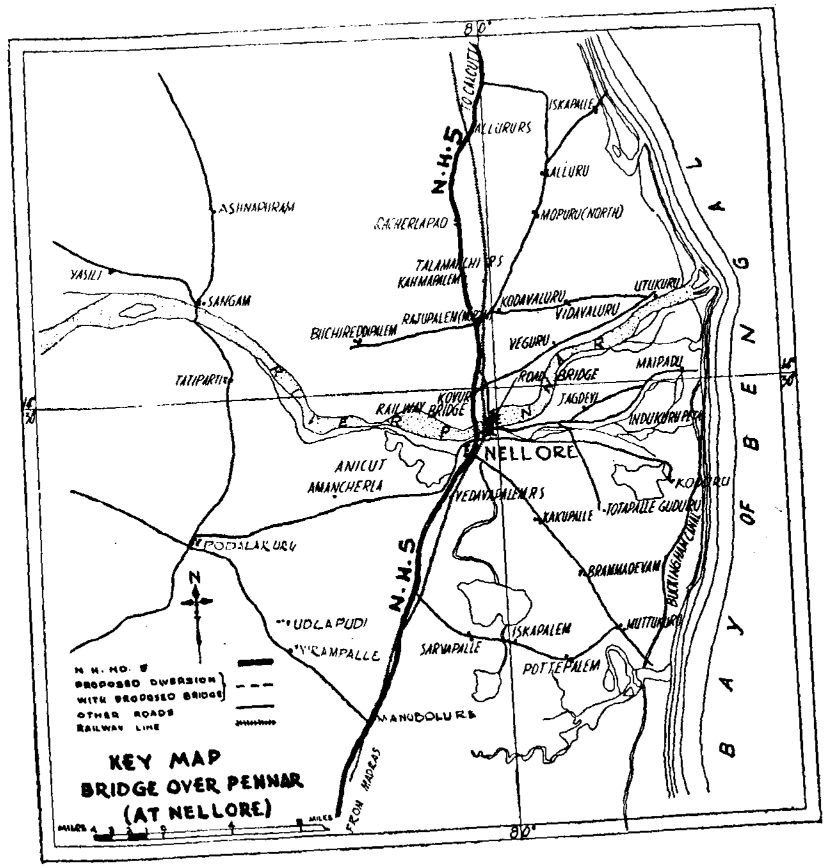


Fig. 1.

4. WATERWAY

The railway bridge is 300 ft upstream of the present road bridge and has a linear waterway of 2,210 ft and the same was adopted for the road bridge. The same number and size of spans, as in the case of the railway bridge, were adopted on account of the close proximity of the railway bridge, at the instance of the railway authorities, to avoid cross currents and development of eddies and bad scours at the piers.

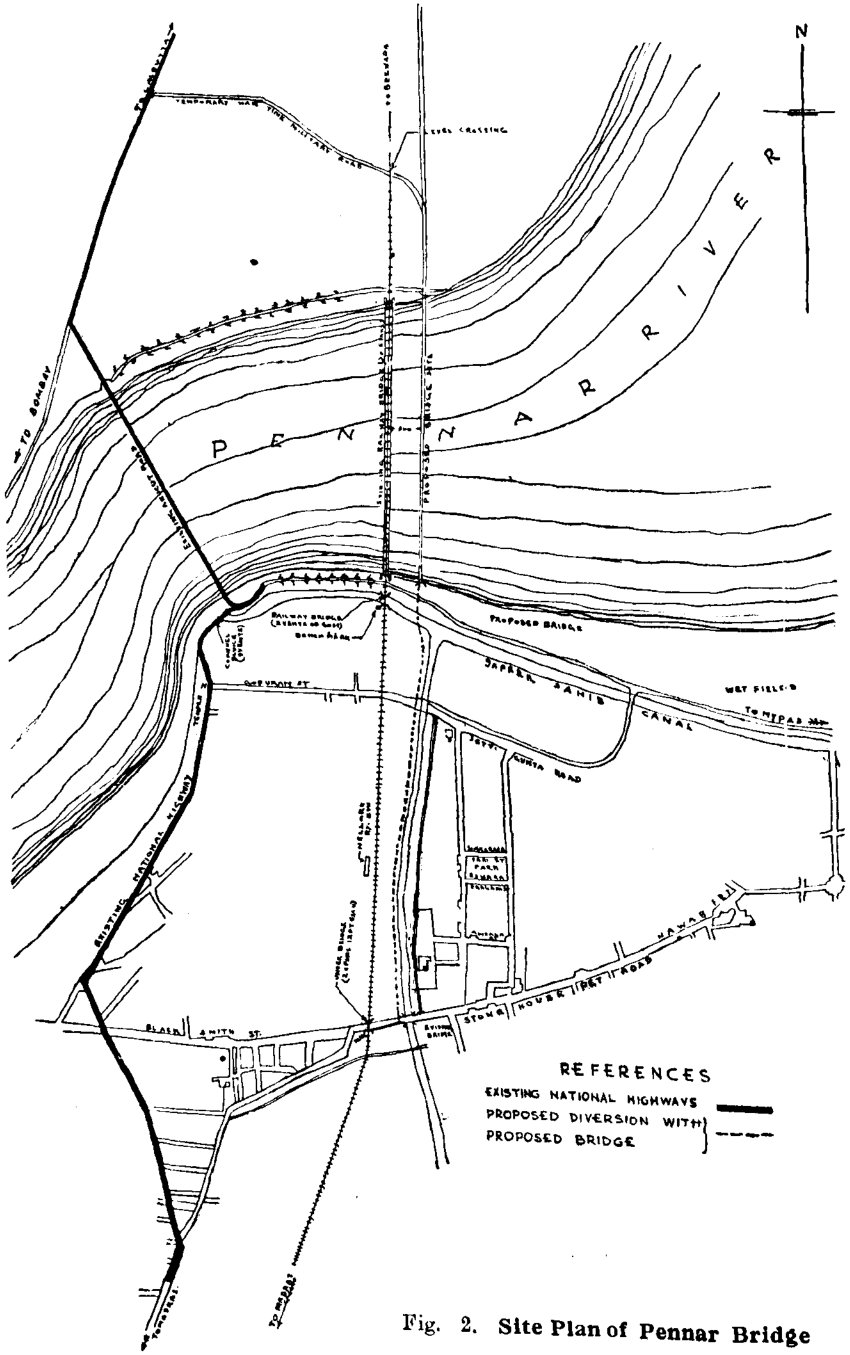


Fig. 2. **Site Plan of Pennar Bridge**

5. TYPE OF BRIDGE

The bridge has 23 spans of 71 ft clear and 10 spans of 61 ft clear, making its total length 2,426 ft between the faces of abutments, Plate I. On account of obvious advantages, viz., cheapness in initial cost and no subsequent maintenance as compared with other alternatives, a reinforced concrete superstructure of Tee-beam-cum-slab design was adopted for this bridge with cantilever footpaths 5 ft wide on either side of the roadway which would serve to segregate pedestrian traffic that is expected to be fairly heavy for reasons of close proximity of the bridge to Nellore town. Advantage was also taken of the footwalks to provide a trough underneath throughout the length of the bridge to house electrical cables and water mains expected to be laid at a later date to serve the extension of Nellore town towards the river.

6. GENERAL DESCRIPTION OF THE BRIDGE

6.1. Salient Features:

(1)	Width of roadway	22 ft
(2)	Width of footpaths on either side	5 ft
(3)	Total number of spans	23 of 71 ft clear 10 of 61 ft clear
(4)	Total length of bridge	2,505 ft
(5)	Type of foundations :	
	(a) Well foundations	for 21 piers
	(b) Open foundations	for 10 piers and abutments
(6)	Size of wells	Three 11 ft square with rounded corners for each pier
(7)	Maximum depth of foundations	42 ft
(8)	Height of piers	30 ft
(9)	Size of piers	6 ft at top for 71 ft spans and 5 ft at top for 61 ft spans with 1 in 24 batter

(10) Type of superstructure — Tee-beam-cum slab simply supported

(11) Details of superstructure :

(a) 4 Tee-beams of 21 in. by 81 in. with 8½ in. thick deck slab for 71 ft spans.

(b) 4 Tee-beams of 20 in. by 62 in. with 9 in. thick deck slab for 61 ft spans.

(12) Design live load :

I.R.C. Class *AA*	Single lane
I.R.C. Class *A*	Double lane

(13) Important levels connected with the bridge :

High flood level	+ 58.00
Ordinary flood level	+ 48.00
Bottom of foundations	− 12.00
Average bed level	+ 30.00
Top of wells	+ 28.50
Top of capping slab	+ 31.00
Top of pier for 71 ft spans	+ 60.10
Top of pier for 61 ft spans	+ 61.64
Top of bed block for 71 ft spans	+ 61.60
Top of bed block for 61 ft spans	+ 63.14
Top of road level	+ 69.83
Top of footpaths	+ 70.58

(14) Wearing coat — 3 in. thick

6.2. Foundations

Laterite rock was available at the site of south abutment, north abutment and piers 22 to 32. Open foundations were, therefore, adopted in the case of both the abutments and piers 22 to 32, Fig. 3. For piers 1 to 21, well foundations were adopted, Fig. 4. Laterite was met with for the first pier at R.L. +7.00 and the wells were founded on laterite. The wells of all other piers were founded on different soils such as yellow clay, sand mixed with clay and disintegrated rock, Fig. 5.

In the case of open foundations, the bottom of foundations was taken 2 ft into laterite rock. The foundation concrete was of mix 1 : 3 : 6 and 2 ft thick.

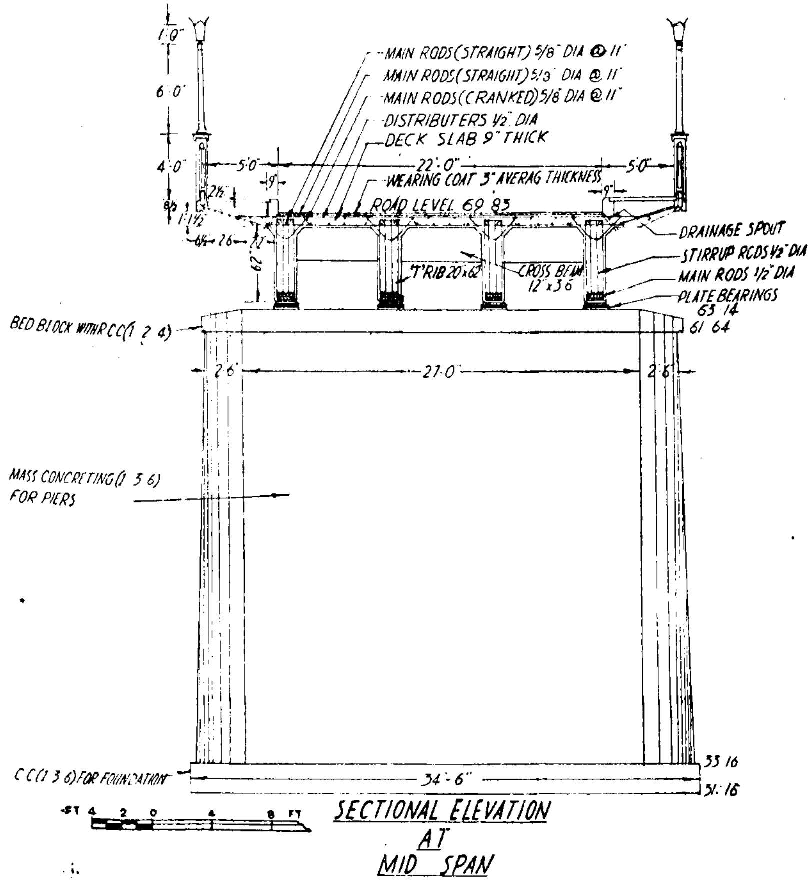


Fig. 3. Sectional elevation showing details of well foundations and superstructure of 61 ft-0 in. (clear) spans

The piers 1 to 21 for which well foundations were adopted have three square wells of 7 ft square (internal) with rounded corners under each pier and were spaced at 14 ft centre to centre of wells. The thickness of steining was 2 ft for the top 30 feet. The steining was of cement concrete 1 : 3 : 6 mix with bond rods and bond plates. Where the thickness increased from 2 ft to 2 ft 6 in. the offset was given on the outside of the well. The wells were provided with R.C. curbs with a cutting edge of 6 in. by 6 in. by ½ in. angle iron at bottom and were sunk to depths as shown in Fig. 5. The bottom, Fig. 6, 6 ft depth of wells was

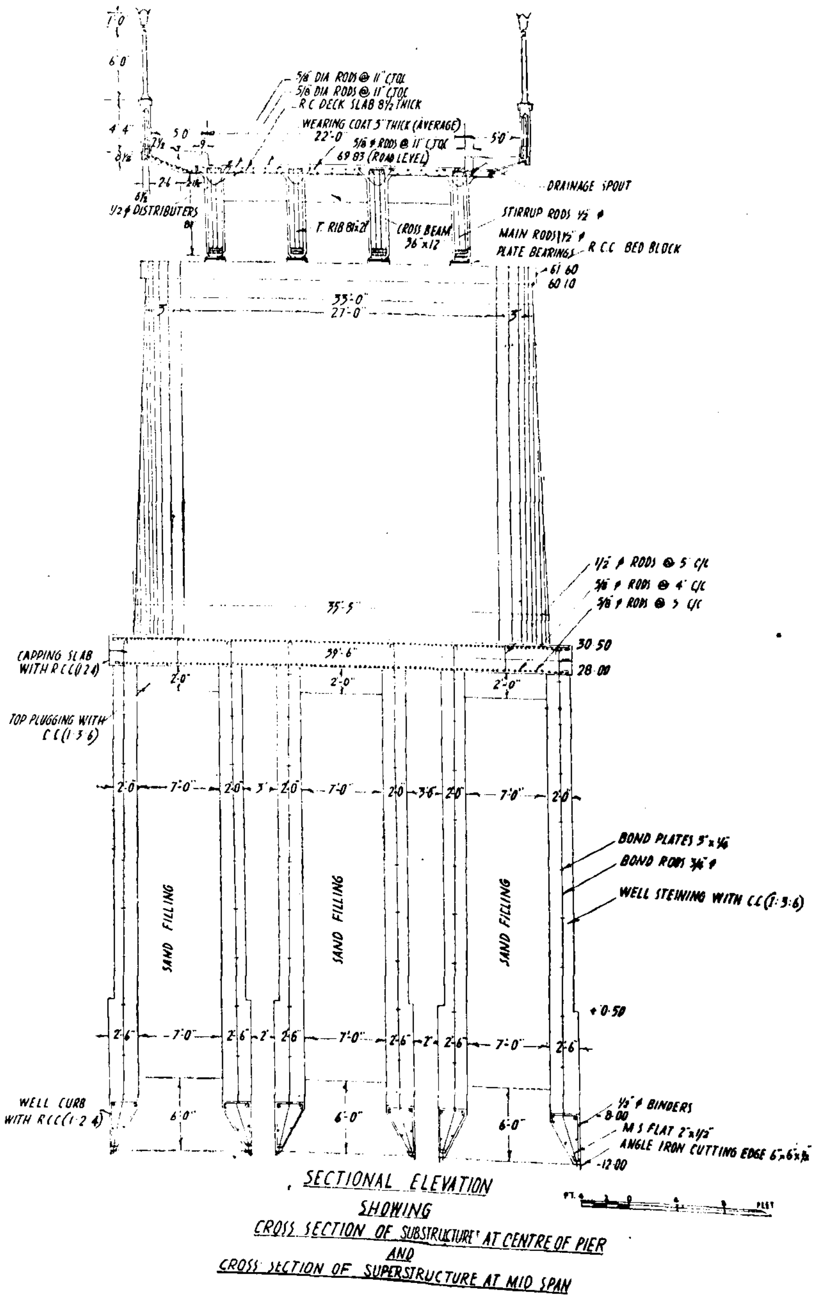


Fig. 4. Sectional elevation showing details of well foundations and superstructure of 71 ft-0 in. (clear) spans

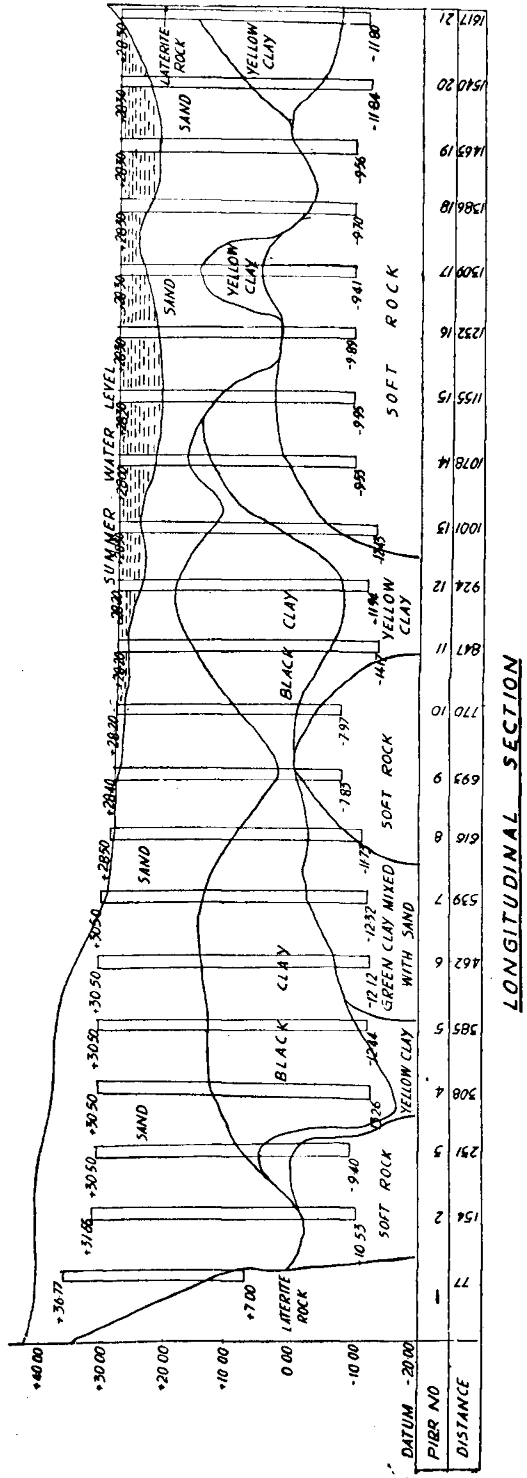
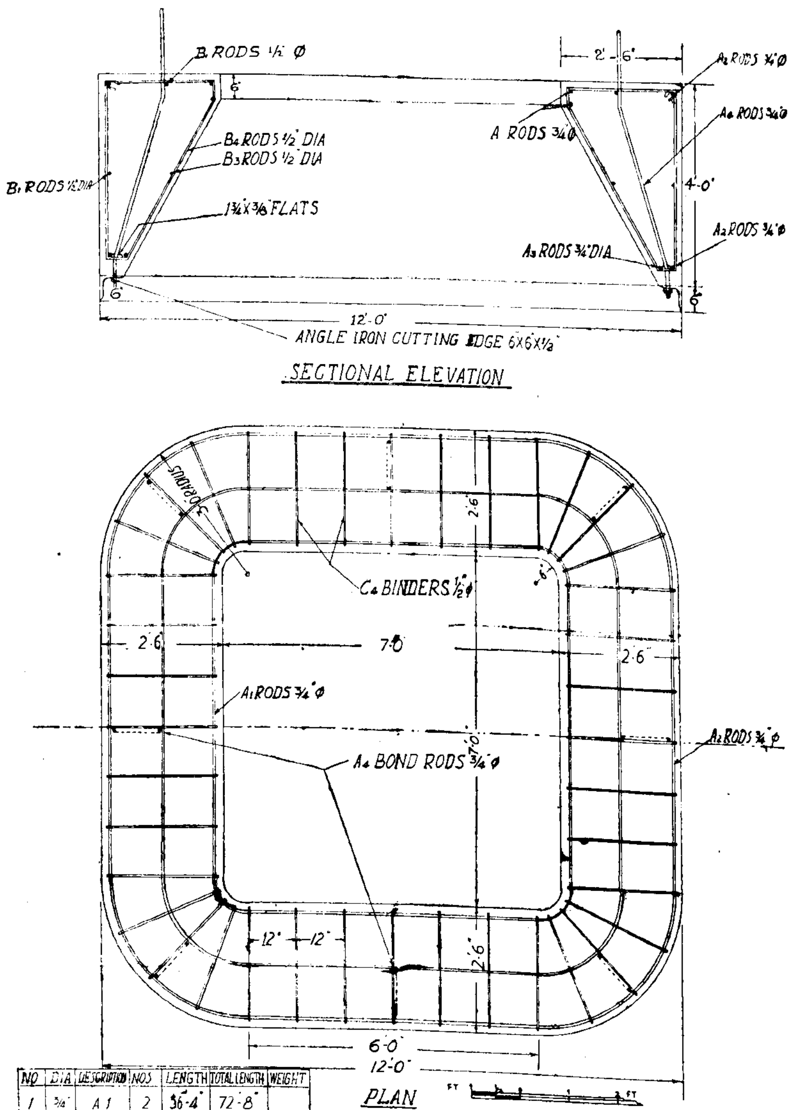


Fig. 5. Particulars of soil met with in well foundations



NO	DIA	DESCRIPTION	NOS	LENGTH	TOTAL LENGTH	WEIGHT
1	¾"	A 1	2	36'-4"	72'-8"	
2	¾"	A 2	2	54'-9"	109'-6"	
3	¾"	A 3	1	52'-0"	52'-0"	
4	¾"	A 4	8	4'-8"	38'-6"	406 LBS
					272'-7"	
5	½"	B 1	1	36'-10"	36'-10"	
6	½"	B 2	1	47'-9"	47'-9"	
7	½"	B 3	1	36'-8"	36'-8"	
8	½"	B 4	40	11'-3"	11'-3"	
9	1¾"X⅜" FLAT		1	43'-0"	571'-3"	582"
					43'-0"	95"
					ADD FOR BOLTS	[illegible]
						683"

Fig. 6. **Details of Well curb**

plugged with cement concrete 1 : 2 : 4. The top 2 ft depth of wells was plugged with cement concrete 1 : 3 : 6. The heart of the wells between the two plugs was filled with sand. Over the wells a continuous capping slab 2 ft 6 in. thick was provided, Plate II.

6.3. Substructure

6.3.1. **Piers** :—The piers are of mass concrete 1 : 3 : 6. The top width of piers is 6 ft for the 71 ft spans and 5 ft for the 61 ft spans. The cut waters of piers are of semi-circular type both on the upstream and the downstream sides. The piers were given a batter of 1 in 24 on all sides. Cement concrete 1 : 2 : 4 mix was laid for one foot width and for half the upstream side of piers to make the piers strong enough to withstand the impact of rolling debris and the velocity of current.

6.3.2. **Abutments** :—The north and south abutments were founded on laterite rock. The abutments have straight returns and are of mass concrete 1 : 3 : 6 mix. For south abutment, the wings are stepped up and founded on higher levels, Fig. 7. Batter of 1 in 12 was given for the outside faces of the abutments and wings.

6.3.3. **Bed Blocks** :—The bed blocks are continuous over the piers and were cast with reinforced cement concrete 1 : 2 : 4 mix and 1 ft 6 in. thickness. The bed blocks project from the pier by 3 in. on all the sides and a drip course is provided at the middle of the projection, Plate II.

6.4. Bearings

6.4.1. Mild steel plate bearings were provided at movable ends and also at fixed ends. All the eight movable bearings were fixed on one pier and all the fixed bearings on the alternate pier.

6.4.2. Originally, provision was made for cast steel roller and rocker type bearings. The cost of these bearings alone was as much as Rs 4 lakhs and it was, therefore, considered that plate bearings would serve the purpose. Accordingly, they were changed to plate bearings with 2 in. thick plates both at the bottom and at the top. This change reduced the cost of bearings to Rs 1.2 lakh.

6.4.3. Two inch thick plates were not available in the market at the time of manufacture. Two 1-in. plates were, therefore, joined together by rivetting and by welding on all the sides to get the required thickness, Plate III.

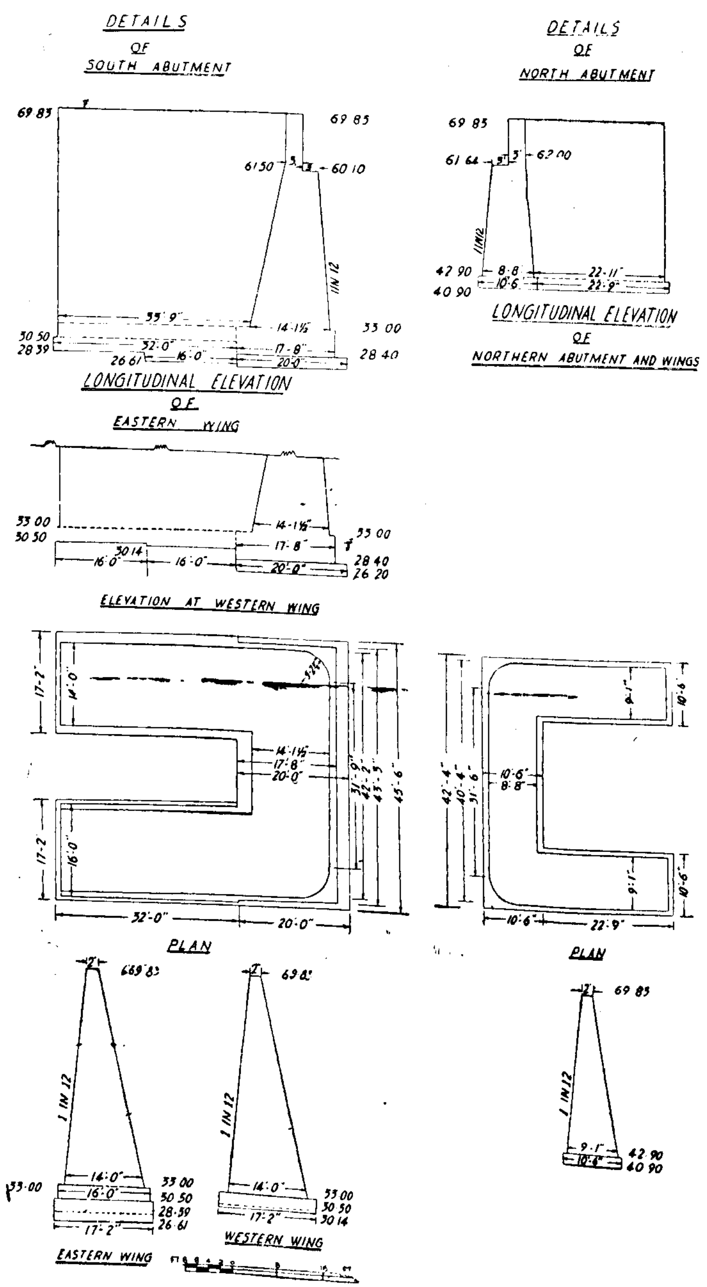


Fig. 7. **Details of abutments and wing walls**

6.5. **Superstructure**

6.5.1. The superstructure consists of reinforced cement concrete Tee-beams and decking. The deck slab is supported on a system of 4 beams spaced at 7 ft centre to centre of beams, Plate IV.

Cross beams are provided at ends and at one-third points of beams to serve as stiffeners to the ribs of Tee-beams, Plate V.

6.5.2. **Expansion Joints:**—The superstructure of each span is completely separated by suitable expansion joints. A gap of 1 in. width has been left right through the superstructure including the wearing surface and hand railing. Copper plates 1/16 in. thick bent as shown in Fig. 8 were provided between the deck slab and wearing surface to cover the gap and the 1-in. gap over the copper sheet was filled with a resilient filler to give a continuous riding surface.

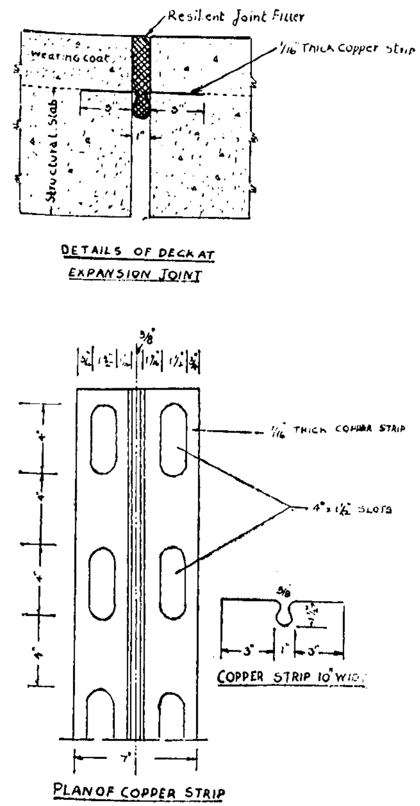


Fig. 8.

6.5.3. **Width of Roadway:**— The clear width of the roadway is 22 ft between the kerbs. Five feet wide R. C. cantilever footpaths were provided on either side of the carriageway and are 9 in. higher than the carriageway.

6.5.4. **Wearing Surface:**— The wearing surface consists of 3 in. thick (average) 1 : $1\frac{1}{2}$: 3 cement concrete laid over the deck slab, the top being properly dressed so as to slope towards the kerbs for drainage purposes.

Tell-tale blocks cast in white coloured cement were left in the wearing coat for measuring the depth of wearing at a later date.

The top width of the block is 1 in. and it is given a slope of 1 : 1 on either side. The depth of wearing can be calculated by measuring the increased width due to wearing, Fig. 9.

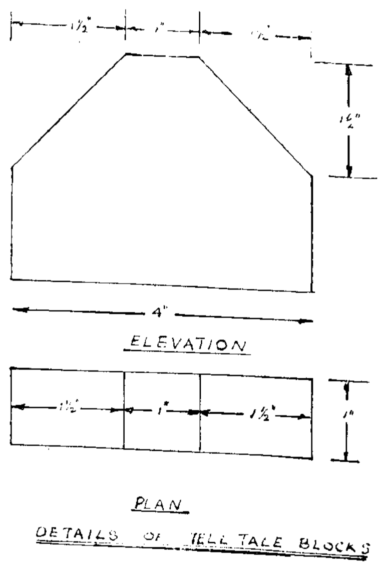


Fig. 9.

6.5.5. **Hand Railing**:—The hand railing consists of precast R. C. panels fixed in R. C. bottom rails, top rails and posts cast in situ. R. C. pedestals were provided at the end of each span to carry lamp posts and to give architectural effect.

Lamp posts are provided on either side of the roadway over each pier. Artistic domes are provided over each post. The cost of electrification of the bridge including the cost of domes was Rs 16,900.

7. DETAILS OF CONSTRUCTION

7.1. General

7.1.1. A special division for this work was opened in November 1949 and tenders were called for. Several engineering firms competed, but since all the quotations were very high, it was decided to execute the work departmentally. The work was started in December 1949. During the course of departmental

execution it was thought expedient to split the work into three parts, namely, foundations, substructure and superstructure, and to invite piecework tenders individually for the three items. A favourable response came at 10 per cent to 12 per cent above the schedule rates and accordingly the work was allotted to three different contractors.

7.1.2. Two permanent theodolite stations were constructed for aligning the bridge axis on either bank. The open foundation work on the northern side was first taken up and the position of the piers was marked with reference to the railway bridge. There was water to a considerable length of the river and the position of the piers on the southern side was fixed by triangulation with a base line of 800 ft on the southern shoal.

7.2. Open Foundations

7.2.1. There was no difficulty in the execution of this work except in the case of pier 22. In the case of south abutment and two piers, excavation and concreting had to be done below water level. As they were not situated in deep bed portion, the work was done without any elaborate shoring and strutting arrangements. Pumps fitted with 10 H. P. engines were used for baling out water.

7.2.2. In the case of pier No. 22, laterite was available at R.L. 31.00 for about 30 ft in length but suddenly sloped down to R.L. 16.00 for the rest of the length. A coffer dam was formed with two rows of sheet piles and the space between the two rows was plugged with clay. The sand overlying the laterite was excavated up to R.L.+16.00. Concrete was laid in this portion up to R.L. 27.00 and over this a continuous slab was laid to the full length of the foundation.

7.3. Well Foundations

7.3.1. The work was started on the southern shoal. The bed of the river up to pier No. 8 was higher than the summer water level. Sand was excavated up to one foot above the water level and R.C. well curbs were cast. In the deep bed portion temporary islands were formed with sand and curbs were cast over the islands. The well curbs were 4 ft high and 2 ft 6 in. thick at top tapering to 6 in. at bottom, Fig. 6. An angle iron cutting edge of size 6 in. by 6 in. and ½ in. thick was provided at the bottom to facilitate sinking. The angle iron was fixed to the bond rods left in the kerbs with bolts and nuts. Teakwood shuttering was used for concreting. Cement concrete 1 : 2 : 4 was used for casting the curbs.

7.3.2. The kerb was sunk and the steining raised in stages of 5 ft over the curbs. Teakwood shutters with sheet lining were used for casting the steining. Timber distance pieces were used to maintain the thickness and straps were fixed around the outer surface of shuttering to prevent bulging of shutters during concreting.

7.3.3. **Sinking:**—Sinking was done by open dredging up to 4 feet. From 4 ft to 15 ft depth sinking was done under water with divers without any equipment. Thereafter, diving helmets were used. Ten hand operated air pumps and 3 air compressors were used for supplying air to the helmets. One air pump operated by manual labour can supply air to two helmets and one air compressor of 110 cu. ft capacity can feed 8 helmets. Two divers can simultaneously work in each well and each sinker can work under water continuously for three hours.

7.3.4. Sinking of wells in sand was an easy job. Difficulty was felt only when boulders were met with in sand. The boulders were met just underneath the cutting edge. Such boulders were broken by using pneumatic drills. The boulders found at the middle of the well were hauled up by using 5-ton differential pulley and tripod.

7.3.5. For most of the wells clayey soils such as black coloured clay, black clay mixed with red laterite gravel, yellow coloured clay, yellow clay mixed with red coloured gravel, were met with just below the sand layer. Sinking in the soils for the first 1 to 2 ft was very slow. With two divers working in one well, sinking in these soils was never more than one foot for 15 days. Pneumatic spades were used to excavate the soil. After sinking 1 to 2 ft in clayey soils, the wells could be dewatered completely by using sump pumps worked on compressed air and thereafter sinking could be done by open dredging. Where this was feasible, four sinkers could easily sink the well 2 to 3 ft per day.

7.3.6. But in some cases open dredging could not be done due to blow of sand during dewatering. In such cases, the wells had to be sunk for another 1 to 2 ft in clay before dewatering. The passages where sand blow occurred were packed with empty gunny bags and steel wedges before the wells were dewatered.

7.3.7. Another difficulty met with was that wells were not going down even though the earth below the cutting edge was excavated 3 to 5 ft below the cutting edge due to skin friction. This was observed from 30 to 40 ft depths. In such cases, loading of wells was tried and was successful in many cases; where loading the well also failed to sink the wells, a small blast of gelignite was given. This proved very successful.

7.3.8. The space between the wells was 3 ft for the top 30 ft and 2 ft below. The earth between the wells got loosened due to this change and there was a tendency for the two outer wells to draw closer towards the central well. This was prevented by keeping distance pieces between the steining while sinking the outer wells.

7.3.9. Borings were taken near all the piers with calyx drill and also with hand boring set before deciding to stop the sinking of wells.

7.3.10. According to the particulars gathered during the investigation, laterite rock was met with at piers 2 and 3 at R.L. 0.00 and -10.00 respectively. But no laterite was met with at those levels. Borings were taken up to -20.00 to see whether any laterite was available at lower depths. Only disintegrated rock was met at R.L. -2.00. The wells were stopped at R.L. -10.00.

7.3.11. In the case of piers 6 and 7, yellow clay was shown at R.L. -12.00. But at that level sand mixed with green coloured clay was met with. Borings were taken to R.L. -20.00 and the same strata was available. The wells were stopped at -12.00.

7.3.12. According to the tender condition a rate of Rs 135 was fixed for sinking the wells per running foot in sandy, clayey and silty soils. The actual cost for sinking the wells in sand was about Rs 50 per running foot. The actual cost for sinking the wells in clayey soils was as much as Rs 400 per running foot. Where it was possible to dewater the well, sinking per running foot was costing about Rs 50 on the average. For soils other than those mentioned above, the contractors were paid on labour basis at the rate of Rs 6 per diving hour including all charges. In disintegrated rock the actual cost for sinking the wells was as much as Rs 500 per running foot.

7.3.13. **Bottom Plug:**—After a well was sunk to the required level, the bottom 6 ft depth was plugged with cement concrete 1 : 2 : 4 with $1\frac{1}{2}$ in. size granite metal. For most of the wells concreting was done under water and in some cases concreting was done after dewatering the complete well.

7.3.14. For concreting under water a steel box with shutters at bottom was used. A hand operated winch and tripod were used for lowering and lifting the box. When the wells were dewatered completely after a period of one week after completing the concreting of all the three wells of a pier, the wells in most cases were found to be watertight.

7.3.15. For some wells water could be completely baled out and concreting was done using a "Tremmie". The Tremmie consists of a hopper connected to a pipe made of steel sheet. The pipe was taken to the bottom of the well which was completely dewatered. Concrete was poured from top through the hopper. It was found that the wells plugged in this manner were not watertight. The small springs at the bottom of the well would pierce through the freshly laid concrete and passages were left even after the concrete had set. In those wells plugging was done for another one foot extra depth under water and the wells were found to be watertight. This method was later abandoned and all the other wells were plugged using the steel box.

7.3.16. Some wells, when dewatered, showed cracks in the corners of the well steining. The cause for the development of these cracks could not be traced. In such cases the wells were plugged with cement concrete 1 : 3 : 6 covering the whole depth of the crack.

7.3.17. **Sand Plug**:—The wells were completely dewatered and sand plugging was done in layers of one foot thickness. First, 1-ft layer of sand was dumped and the same was levelled and watered till the sand became completely wet and the layer was consolidated using iron rammers before the second layer was taken up.

7.3.18. There was some controversy regarding the method adopted for sand filling. It was argued that sand would get maximum compaction when filled under water and that there was no need to do the sand plugging in one foot layers.

7.3.19. A simple test was conducted with a steel container. First sand was put under water to a depth of two feet. The sand was then removed and the same quantity was put in layers well watered and compacted. It was observed that the settlement was more when it was done in layers and so this method was adopted for all the wells.

7.3.20. **Top Plug**—The top 2 ft depth of wells was plugged with cement concrete 1 : 3 : 6. The variation in the top of wells was also made up in the top plugging.

7.3.21. **Capping Slab**—The capping slab covered the three wells of a pier and was 39 ft 6 in. in length, 11 ft 6 in. in width

and 2 ft 6 in. in depth. The capping slab projected from the wells by 3 in. on all sides. Teakwood shuttering with G.I. sheet lining was used for concreting the capping slab (1 : 2 : 4), Plate II.

7.4. **Substructure**

7.4.1. **Piers**—The piers were raised in stages of 5 ft each. Teakwood shuttering with G.I sheet lining was used for concreting the piers. The shuttering for the first five feet height was fixed with timber distance pieces on the top and external timber struts. After the first stage of concreting, the top struts were removed and another stage fixed over the first stage shutters. Shutters remained for the complete height of the pier. Concreting was done in layers of 6 in. and consolidated with iron rammers. The piers were provided with R.C. continuous bed blocks on top, Plate II.

7.4.2. **Abutments**—Abutments and wing walls were also of mass concrete in cement mortar 1 : 3. The abutments were raised in stages of 3 feet.

7.5. **Superstructure**

7.5.1. **Centering**—The centering to support the formwork of the Tee-beams and slab consisted of casuarina posts of 30 ft to 35 ft length and 6 in. in diameter at bottom and 4 in. diameter at top. Horizontal stiffeners were provided in both directions at 4 ft 6 in. spacing. Diagonal bracings were also provided, Plate VI. Also see Appendices 1 and 2.

7.5.2. Generally, there are floods in the river Pennar during the months of July to December. To augment the progress of the work during this period, '*N*' type trusses made out of Callendar-Hamilton bridge parts that were used for the construction of the bridge across Pugalur were obtained and were assembled to 77 ft lengths. One of the trusses was test loaded with sand bags equivalent to the weight of one beam and deck slab over the beam. The truss showed a deflection of 3 in. and it was decided not to use these trusses.

7.5.3. **Main Reinforcement**—$1\frac{1}{2}$ in. diameter rods were used for the main reinforcement. The rods were supplied in standard lengths of 40 ft to 42 ft and to get the full length of the reinforcement rods welding was resorted to. The ends of the rods were cut to '*V*' shape, Fig. 10, and butt welding was carried out with No. 8 gauge electrodes for filling and No. 10 gauge electrodes for finishing. The welded joints were tested and it was found that failure always took place outside the limits of the weld, Table 2. The position of the welds was arranged in such a way that no two welds were in the same vertical plane and the welds were staggered by 2 ft and more. Appendices 3 and 4 may be referred to for other details.

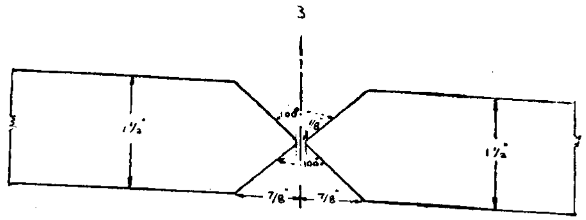


Fig. 10.

Details of welding joint

7.5.4. **Concreting**—The ribs of the Tee-beams and the cross-beams were concreted first up to 9 in. below the bottom of the deck slab. All the four beams were first concreted. This took two days at the rate of two beams per day. After concreting the beams, reinforcement for the deck slabs was tied and the deck concreted. This took another 1½ days. The total quantity of concrete in each span of 71 ft span was 5,700 cubic feet. Two Millar concrete mixers were used for mixing the concrete (Appendix 5 for other details).

7.5.5. Sand used for concreting was never dry and due allowance was made for bulking of sand. A graduated jar was used for measuring the bulkage. Test blocks were taken for all the spans and they gave very good results, Table 1.

7.5.6. **Hand Railing**—The panels were first cast using teak-wood moulds with G.I. sheet lining, Fig. 11. They were later kept in position and the posts, bottom rails, and top rails were cast in situ.

7.5.7. Originally, cement concrete 1 : 2 : 4 with ¾ in. granite jelly was provided for the posts, bottom rails and top rails. When hand railing over the first five spans was laid it was observed that the finish was not up to the mark and the corners were pealing off. So it was considered desirable to have a richer mix. A richer mix of 1 : 1½ : 3 with ½ in. size granite jelly was used for the rest of the hand railing and the finish obtained was good.

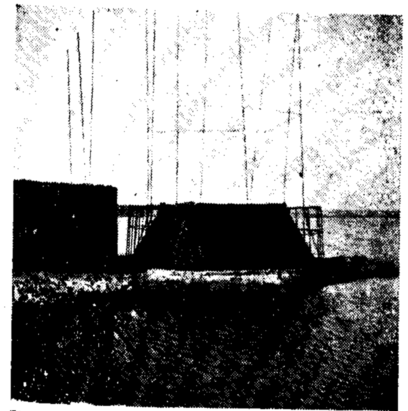


Photo 1. **Reinforcement for well kerb 6″×6″×1/2″. angle iron can be seen at bottom**

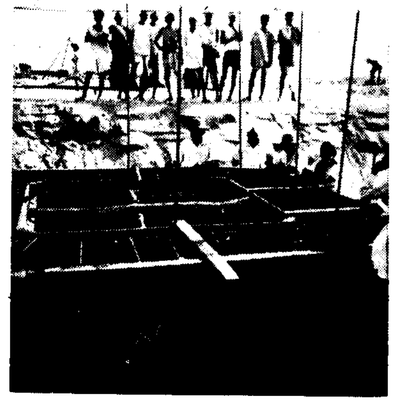


Photo 2. **Shutters for well curb**

Photo 3. **Diver with helmet and dress getting into the well**

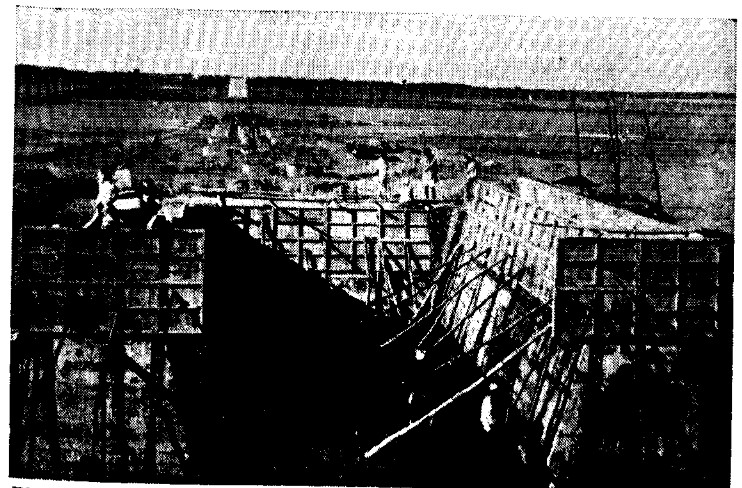


Photo 4. **South abutment in progress. Shuttering fixed for wings and abutment can be seen**

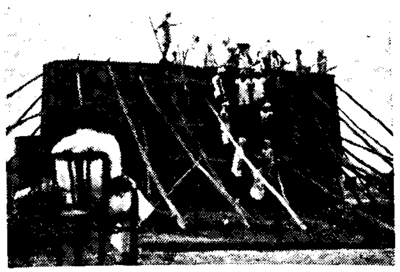


Photo 5. **Pier under construction. Shuttering for piers can be seen**

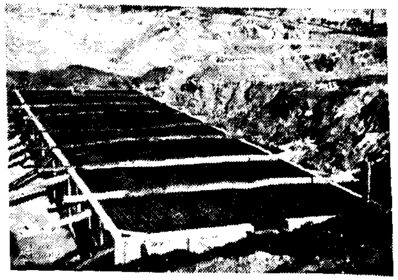


Photo 6. **Capping slab over wells. Reinforcement and shuttering can be seen**

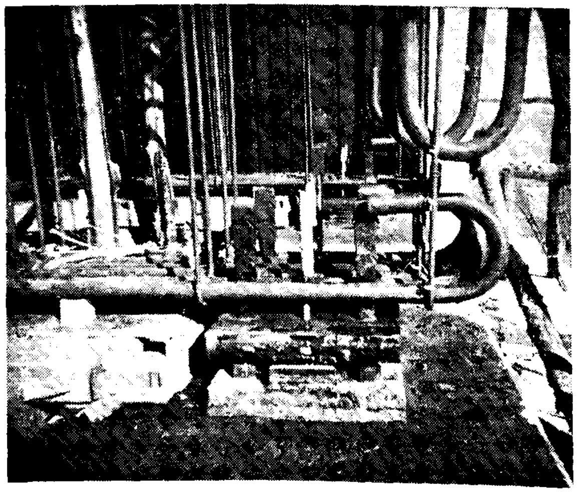


Photo 7. **Plate bearings Bottom plate fixed to bed block**

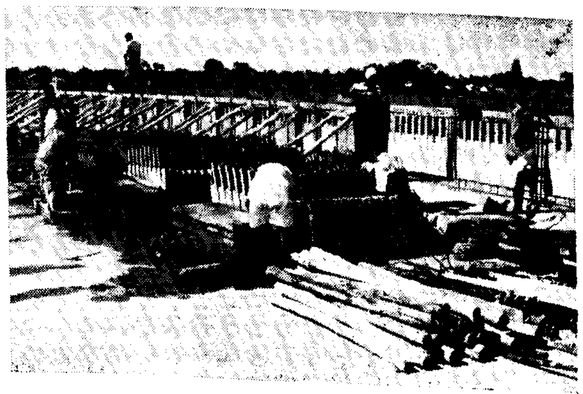


Photo 8. **Construction of Hand Railing Precast panels kept in position. Centering fixed for concreting posts**

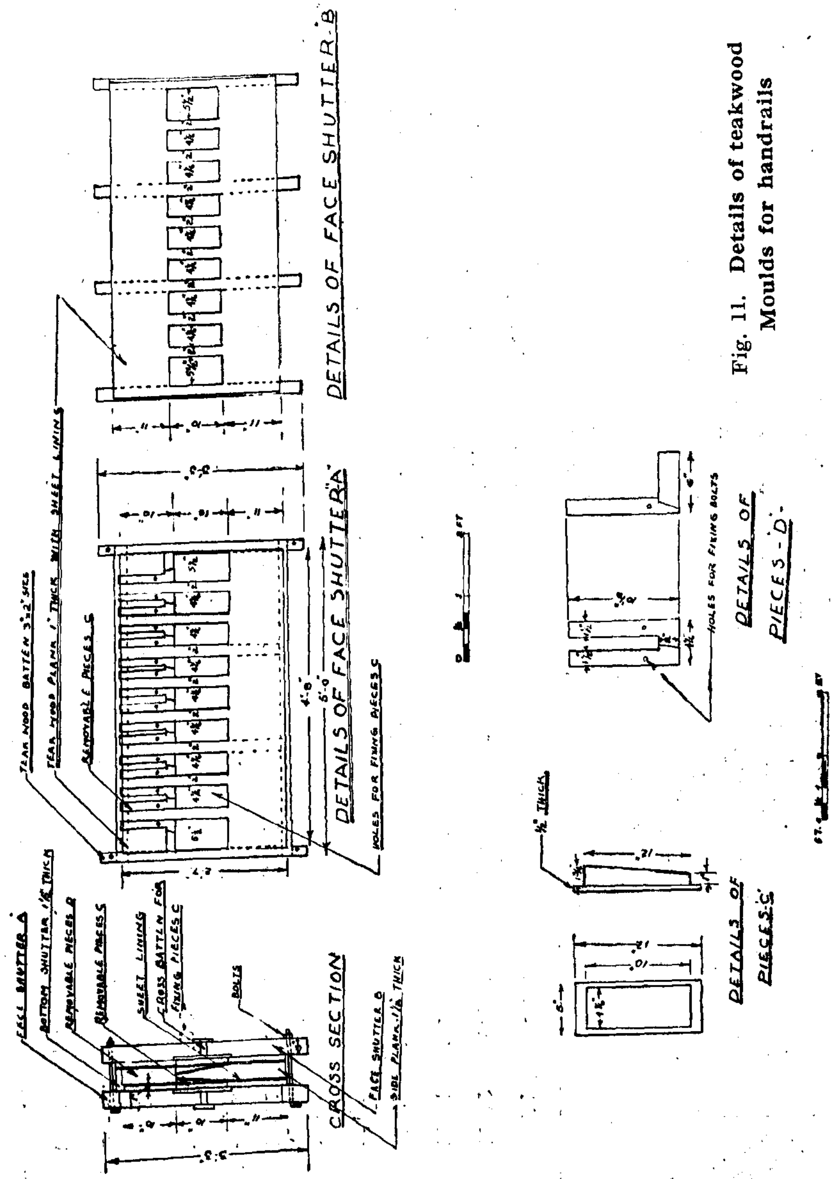


Fig. 11. **Details of teakwood Moulds for handrails**

TABLE 1

Compression tests of concrete cubes
Size of test cubes 6 in. × 6 in. × 6 in.
Areas under compression = 36 square inches Mix. 1 : 2 : 4

S. No.	Mark on specimen		Water-cement ratio	Weight in lbs	Load at first crack		Crushing load		Remarks
					Pressure in tons	Lb per sq. in.	Pressure in tons	Lb per sq. in.	
1	12 A1	9-1-54	0.50	18	55	3422	90	5600	
2	12 A2	9-1-54	0.45	18	60	3733	89	5538	
3	12 B1	10-1-54	0.50	18	54	3360	72	4480	
4	12 B2	10-1-54	0.45	18	50	3111	85	5289	
5	12 C1	10-1-54	0.50	18	70	4355	84	5226	
6	12 C2	10-1-54	0.45	18	50	3111	79	4915	
7	12 D1	9-1-54	0.50	19	56	3484	70	4355	
8	12 D2	9-1-54	0.45	18	35	2178	92	5724	
9	12 S1	12-1-54	0.45	19	72	4480	96	5973	
10	12 S2	12-1-54	0.45	18	73	4542	97	6035	
11	12 S3	13-1-54	0.45	18	37	2302	67	4169	
12	12 S4	13-1-54	0.45	18	28	1742	67	4169	

TABLE 2

Tension tests of welded rods

S. No.	Diameter of rod in.	Yield point		Ultimate		Remarks
		Total load	Tons per sq. in.	Total load	Tons per sq. in.	
1	$1\frac{1}{2}$	28.20	15.96	47.00	26.60	Specimen failed outside the weld
2	$1\frac{1}{2}$	30.10	17.03	49.43	27.97	Do.
3	$1\frac{1}{2}$	36.20	20.50	47.34	26.80	Do.
4	$1\frac{1}{2}$	31.00	17.60	48.20	27.30	Do.

8. ACCESSORIES

8.1. **Drainage Spouts:**—Drainage spouts of cast iron with gratings at the entrance were fixed near the kerbs on the roadway, Fig. 12. They were spaced at 14 ft 6 in. on either side of the roadway. The wearing surface was sloped down towards

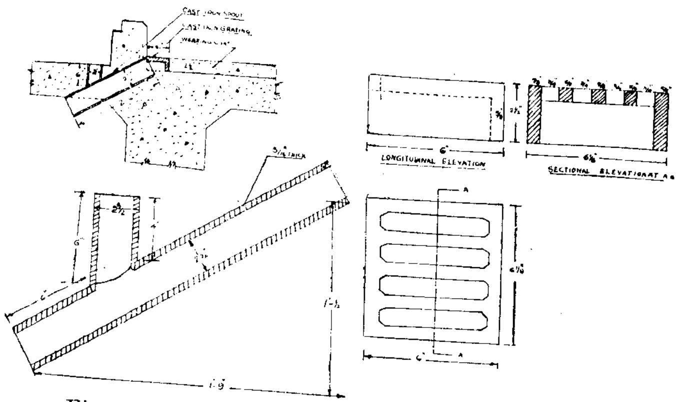


Fig. 12. **Details of drainage spout and grating**

these outlets. It is now observed that the gratings are getting filled up with road dust and constant clearing of these gratings is required for their effective functioning.

8.2. Perhaps this could be avoided by fixing the gratings vertically flush with the kerbs at the road level as shown in Fig. 13.

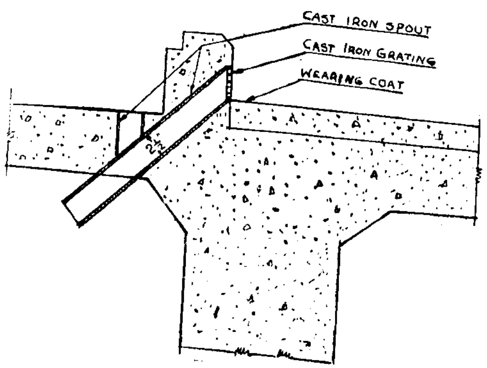


Fig. 13.

8.3. **Scour Meter.** An automatic scour meter was installed at pier No. 20 to record the scour depth at high floods. The device adopted is a very simple one and is similar to the type being adopted in Australia.

8.4. The scour meter consists of 4-in. diameter pipes with bottoms set at different stages on the well depth. $1\frac{1}{4}$-in. diameter probe rods connected with light flexible steel wire are left in these pipes. The wire rope is wound round a drain fixed in a winch box fixed over the pipe on the top of the pier. Any scour below the bottom of the pipe will be revealed by the downward movement of the rod and the wire which will be recorded by the gauges fixed to the winch box. Two such pipes were fixed on the upstream side of pier No. 20. The bottoms of the pipes were left at reduced levels + 22.00 and + 12.00. The probe rod left in the first pipe will record the scour depth up to + 12.00 and the second one up to + 2.00. The dials fixed over the winch box are calibrated to show the reduced levels of the scour depth, Plate VII.

8.5. **Approach Slab**—Reinforced concrete approach slabs were provided on both the abutments to relieve the pressure on the abutments due to live loads. The approach slabs rest on the backing wall for 3 ft and the balance 9 ft on the soil in between the wings. It was felt that there might be slight downward movement in the slab when heavy vehicles passed over it and provision must be made for this. A semi-circular hinge, Fig. 14, was provided with $\frac{1}{4}$ in. bitumen seal over the decking wall for this purpose.

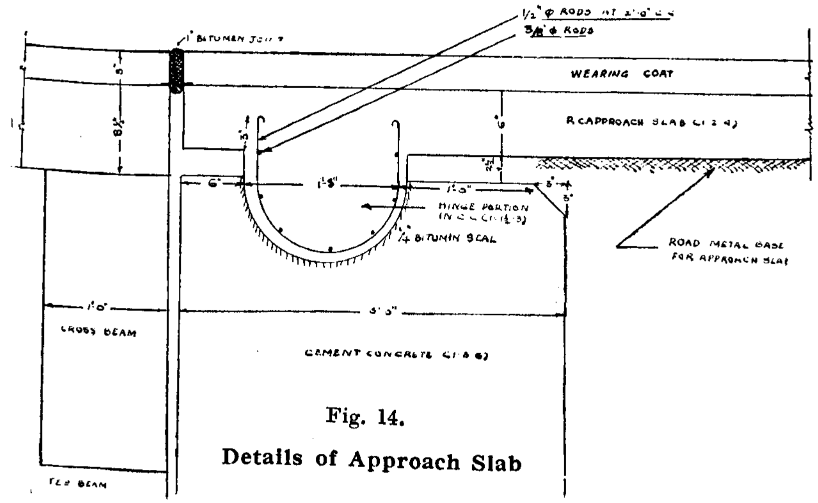


Fig. 14.

Details of Approach Slab

8.6. **Marble Tablets.** Two pedestals of size 5 ft 9 in. wide and 4 ft 4 in. high with hand railing in between them were provided over each wing of the abutments. Eight marble tablets of size 4 ft 3 in. by 2 ft 3 in. with all the particulars of the bridge inscribed thereon were fixed on these pedestals.

8.7. **Guide Posts and Railing.** Continuous type of guide posts and railing were provided for the approaches to the bridge. The formation width of the embankment of the approaches was 38 ft and the posts and railings were fixed at the ends of the formation. Where no revetment was constructed for the slopes of the embankment, deep scours were forming due to rains and the guide posts were slipping and falling. This could be avoided by fixing the guide posts at 16 ft from centre of road leaving 3 ft beyond the posts on either side.

9. GENERAL

9.1. **Quantities involved**

Cement	5,625 tons
Steel	1,045 tons
$1\frac{1}{2}$ in. size granite metal	4,600 units
$\frac{3}{4}$ in. size granite metal	2,100 units

9.2. Granite metal $1\frac{1}{2}$ in. size and $\frac{3}{4}$ in. size was supplied departmentally and through piece workers. A stone crusher was installed at the site of the work. Boulders were got through piece workers and were crushed to the required size at the site of the work.

9.3. A tramway track was laid from the bank to the river bed for a length of 4 furlongs and tipping wagons were used for conveying the materials to the work site.

9.4. **Floods**:—During the month of May 1952 there was a heavy cyclone and consequently there was a heavy unexpected flood in the river. Machinery and materials worth about Rs 2 lakhs were either submerged or washed away. The flood water subsided after 15 days. Most of the teakwood shutters were found lying on the banks beyond 10 miles from the bridge site.

9.5. The whole area of about 300 ft upstream and 300 ft downstream of the bridge was probed to locate the position of the heavy machinery. All the heavy machinery such as compressors, mixers, pumps, etc., were found at th ame places

where they were stationed but were found sunk to depths of 10 ft to 20 ft into sand. Helmet divers were engaged to remove the sand over the plant and by the side of the plant and the machines were then hauled up using tripods and differential pullies. All the heavy machinery and most of the small machinery was salvaged. The cost for salvaging and repairing was as much as Rs 30,000.

10. COSTS

10.1. The cost of the various components of the bridge was:

Foundations	Rs 12,50,000
Substructure	Rs 5,00,000
Bearings	Rs 1,20,000
Superstructure	Rs 14,30,000
Total cost	Rs 33,00,000

10.2. The cost per running foot of linear waterway of the bridge was Rs 1,564.

10.3. The cost per running foot of deck slab of the bridge worked out to Rs 124.

10.4. The cost per sq. ft. of the elevation area comprised by the formation level and bottom of foundation was Rs 21.31.

10.5. The cost per cubic foot of the enclosed volume worked out to Rs 0.62.

10.6. The cost of cement and steel at the work site was Rs 83.75 and Rs 400 per ton respectively.

11. LOAD TEST OF THE SUPERSTRUCTURE

11.1. **Object**

11.1.1. As per Indian Roads Congress Code (1937) clause D. 29 and as per recommendations of the Consulting Engineer (Road Development), all major bridges are to be tested before opening them for traffic. The main objects of the test are:

(a) To subject the various members of the structure to the designed stresses (bending stresses) to verify the soundness of the construction.

(b) To verify the correctness of the design assumptions, for example, the distribution of the various wheel loads among the beams.

11.1.2. These are two distinctively different objectives, one to check whether the work in the field has been carried out to the correct specifications or not, while the other is to check how far the assumptions made in the design of the bridge are correct.

11.2. General

11.2.1. As there is no direct method of measuring stresses, the latter can be checked by the measurement of strains. This can be observed by measuring deflections at critical points.

11.2.2. The load test was conducted on span number 22 of the bridge during the period between 10th and 15th December 1954.

11.3. Preliminary Arrangements

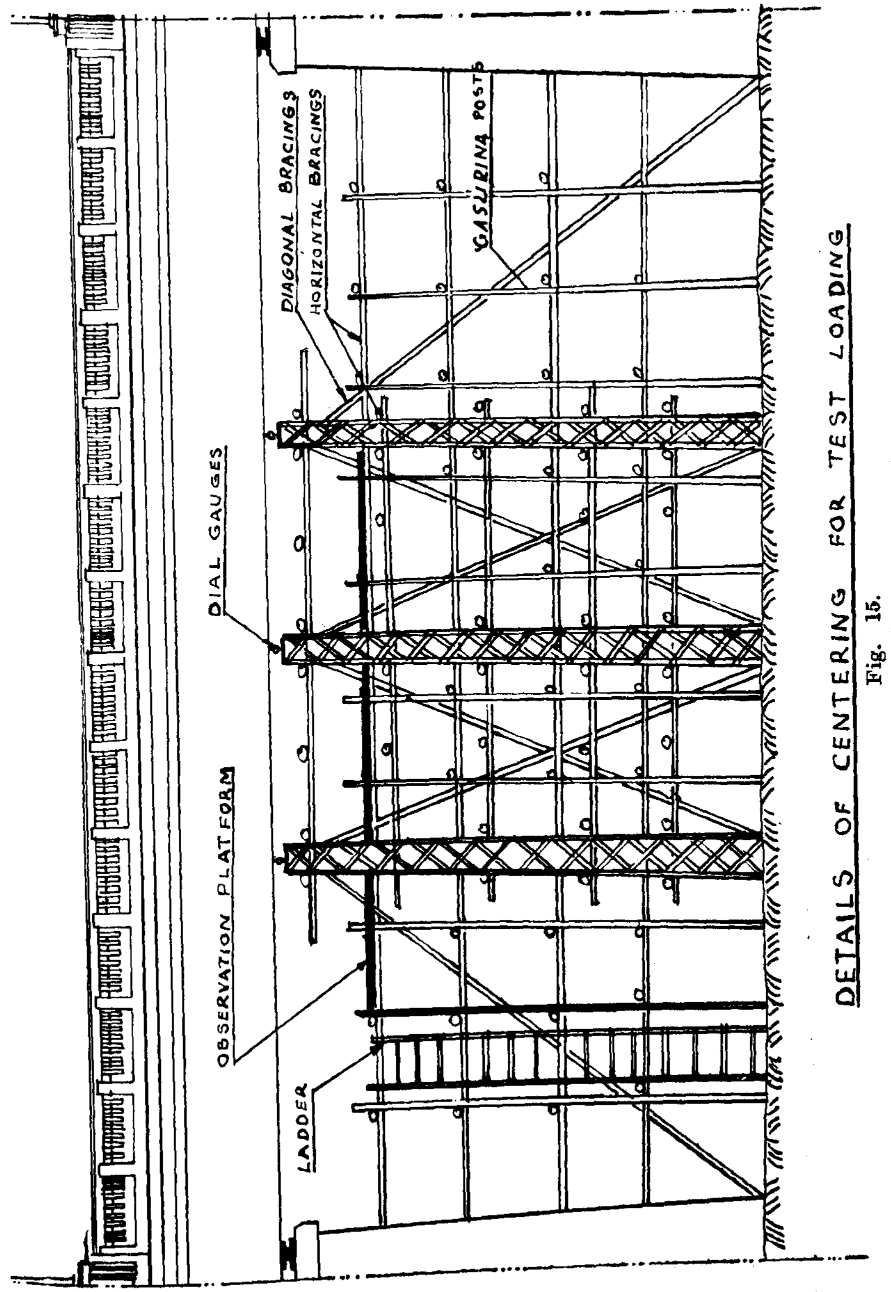
11.3.1. The bed under span No. 23 was rocky. Two independent stagings were erected, Fig. 15, under the span with casuarina posts, one for fixing the deflectometers under the beams and the other for the persons to record the measurements.

11.3.2. The staging for fixing the dial gauges consisted of casuarina posts 4 in number under each beam at the centre and at one-third points. They were taken upto 9 in. below the bottom of beams and the four numbers were rigidly interconnected with bracings. There were 12 such sets and these were interconnected with horizontal stiffeners and inclined posts. Over these vertical posts teakwood planks were fixed. Glass plates were kept under the beam to give even surface and the deflectometers were fixed to teakwood blocks touching the glass plates.

11.3.3. The observation platform was erected 6 ft below the bottom of Tee-beams. This consisted of casuarina verticals, horizontal bracings and inclined posts independent of the staging erected for the dial gauges. Mango planks were fixed for the people to move on this staging to record measurements.

11.4. Loading

11.4.1. As no vehicle equivalent to the one for which the structure was designed was available, alternative loading which gave similar effects was adopted.



DETAILS OF CENTERING FOR TEST LOADING

Fig. 15.

11.4.2. **Loading Series**—(1) Single lorry fully loaded (total load 6 tons) placed symmetrically to the span, the hind wheels of the lorry being over the first cross beam.

(2) The same operation at the centre of the span.

(3) The same operation at the second cross beam.

(4) Single lorry fully loaded placed eccentrically with reference to the centre line of the bridge on the downstream side, the hind wheels of the lorry being over the first cross beam.

(5) The same operation at the centre of the span.

(6) The same operation at the second cross beam.

(7) Operations 4, 5 and 6 repeated with the lorry placed eccentrically with reference to the centre line of the bridge on the upstream side.

(8) The above operations repeated with two lorries.

(9) The above operations repeated with 10-ton road rollers.

(10) The downstream end girder loaded to the full working load by sand bags.

(11) The upstream end girder loaded to the full working load by sand bags.

(12) Both the central beams loaded to the full working load by sand bags.

11.4.3. In the design, the end beams were assumed to take all the dead loads on the cantilever plus half the deck between that beam and the interior beams, while for the interior beams the deck between the centres was directly carried by it. This gave a load of 3,970 lb per rft for each end beam and 2,800 lb per rft for each interior beam including the self weight of the rib. This would give a maximum bending moment of 1,213 foot-tons on the end beam and 856 foot-tons on the interior beam. The ratio of the dead weight carried by the interior beams to the end beams worked out to $1 : 1\frac{2}{5}$.

11.4.4. In the case of live loads due to class A or class AA, the deck slab was assumed to be simply supported on the ribs (the latter again assumed as inelastic) and the reaction on the girder from each wheel or track was thus calculated for the worst position of loading. On this assumption the end beams were subjected to maximum bending moments due to class A while

class AA gave the maximum bending moments for the interior beams. The bending moments on account of these were 287 ft-tons for the end beams and 678 foot-tons for the interior beams, Appendix 6. The ratio of $L.L.$ bending moments between interior and end beams was $2\frac{1}{3} : 1$.

11.4.5. To get the same bending moment due to live loads the end beams were loaded with a uniform distributed load of 0.42 ton per rft, and for the interior beams 1 ton per rft for each beam. So the end beams were loaded to 31 tons each and the interior beams to $2 \times 74 = 148$ tons on the two beams.

Readings of the 12 gauges were recorded for each set of loading.

11.5. **Observations**

11.5.1. From the reading of the dial gauges it was observed that the deflection under the end beam was 0.1 in. and for the middle beam it was 0.11 against the calculated deflection of 0.1 in. and 0.24 in. respectively, Appendix 6. Adding the observations for the four beams, the actual deflection was $\frac{2\times 0.1+2\times 0.11}{2\times 0.1+2\times 0.24} = 62$ per cent of the calculated deflection. No cracks were seen under this load, and on removal of the load there was full recovery of deflections. Hence the bridge could be considered as safe.

11.5.2. From the observed deflections it was seen that for all symmetrical loading with respect to the centre line of the bridge, irrespective of the position of the wheels, all beams carried practically equal loading. When the loads were eccentrically placed with respect to the centre line, it was always the end beam on the side of the load that took more load than all the other beams irrespective of the position of the wheels. This was well demonstrated in the case of one or two rollers placed symmetrically and eccentrically.

11.5.3. When the distributed load was placed on the end beams only and there was no load on the interior beams, it was seen that the latter took 44 per cent of the whole load. Again, when the end beams were loaded 31 tons each and interior beams 74 tons each, it was seen that all beams carried equal loads, i.e., 52 tons each.

11.5.4. A similar distribution was observed in the case of eccentricity of two rollers. Here two wheels out of the four were placed right over one interior beam and only one wheel on the outer beam. But the load carried by the outer beam on the side of eccentricity was 33 per cent more than one wheel load, and the first interior beam, though two wheels were placed over it, actually carried only 12 per cent over one wheel load. The other outer beam carried 70 per cent of the one wheel load though no wheel load was near it.

11.5.5. With different eccentricities the load taken by each beam was observed and this was made use of to check the "Courbon's eccentric factor":

$$A = \frac{6}{n+1} - \frac{e}{\lambda}$$

A = eccentric factor

n = number of girders

e = eccentricity of the loads

λ = average spacing of the girder.

By making use of these observations the eccentric factors for class AA and class A loadings for the outer beams were as follows:

	e	A
Class AA	27 in.	0.4
Class A	22 in.	0.31

Class AA

$$\text{B. M. for the whole bridge} = 2 \times \frac{38.55}{39.88} \times 678 = 1{,}305 \text{ ft-tons}$$

$$\text{Outer beam} = 1.4 \times \frac{1305}{4} = 457 \text{ ft-tons.}$$

Class A – Two lanes

$$\text{B.M. for the whole bridge} = 4 \times \frac{287.4}{0.9048} = 4 \times 318$$

$$= 1{,}272 \text{ ft-tons.}$$

11.5.6. So the outer beams would actually be subjected to a bending moment of 457 ft-tons against the designed bending moment of 288 foot-tons, i.e. 59 per cent under design, and the interior beams no doubt over designed by 100 per cent. This is as far as live loads are concerned.

11.5.7. But in the present design luckily due to the heavy dead weights on the cantilevers and due to the design assumptions that all the cantilever dead loads are carried by the end beams alone, the total dead load and live load results in the end

beams were not adversely affected. As per design, the bending moments were:

	Outer beams	Inner beams
	Ft–tons	Ft–tons
Dead load	1,213	856
Live load	287	678
Total designed for	1,500 ft tons	1,534 ft tons.

11.5.8. Against these the total bending moments carried by the beams based on the tests were:

	Outer beam	Inner beam
	Ft–tons	Ft–tons
Dead load	1,035	1,035
Live load	457	339
Total:	1,492 ft tons	1,374 ft tons.

11.6. Conclusions

11.6.1. The actual deflections under the designed loads were of the order of 62 per cent of the calculated deflection and there were no cracks under the designed loads. Hence the structure as built was according to specification.

11.6.2. It was seen that the end beams carried by far the greater loads over the interior beams due to live loads. Though in the present structure the end beams were not underdesigned due to the presence of heavy cantilever dead loads, the usual assumption would result in the underdesign of the end beams in a 4-beam structure without footpaths.

11.6.3. It is safe to consider all dead loads (symmetrical to centre line of the bridge) equally distributed among all the beams. For live loads due to transverse stiffness, the end beams always carry greater loads than the interior ones when the loads are eccentric and it is imperative to provide for this. The Courbon's eccentric factor $A = \frac{6}{n+1} \frac{e}{\lambda}$ for the end beams is on safe side.

Appendix 1

DETAILS OF SCAFFOLDING

1. There were 600 posts in all for the scaffolding of one span comprising of :

   (a) 210 verticals 4 inches to 6 inches diameter and 30 to 35 feet height placed at 3 feet 6 inches spacing with 21 rows along the span and 10 rows across the span.

   (b) 210 horizontal stiffeners of 3 inches to 4 inches diameter at 4 feet 6 inches spacing along the span.

   (c) 140 horizontal stiffeners 3 inches to 4 inches diameter placed at 4 feet 6 inches spacing across the span.

   (d) 40 numbers inclined braces.

2. The total weight of posts was 70 tons.
3. The total cost was Rs 6,000.
4. There were four such sets and one extra set for replacement.
5. The total cost of scaffolding was Rs 30,000.
6. It took 6 days for erection and 4 days for dismantling and cost Rs 600.

Appendix 2

CENTERING

1. The total cubical contents of concrete in each span was 5700 cubic feet.
2. The total surface area covered for each span was 5750 square feet.
3. The cost of centering per set was Rs 50,000.
4. There were two such sets costing Rs 1,00,000.
5. It took 4 days for putting up centering and three days for removal and cost Rs 2,000.

Appendix 3

GRILL WORK

1. The total quantity in each span was 24.3 tons comprising of :

   (a) 14.7 tons of $1\frac{1}{2}$ inch diameter rods.

   (b) 4.4 tons of $\frac{5}{8}$ inch diameter rods.

   (c) 3.8 tons of $\frac{1}{2}$ inch diameter rods.

   (d) 1.31 tons of $\frac{3}{8}$ inch diameter rods.

   (e) 0.09 tons of $\frac{1}{4}$ inch diameter rods.

2. It took 7 days for bending and 3 days for tying the grills.
3. Cost was Rs 2,000 per span.

Appendix 4

WELDING

1. Two men can cut 5 joints to shape.
2. Metrovick electrodes 8 Nos. of 8 gauge for filling and 4 Nos. of 10 gauge for finishing were used for every joint.
3. It took 50 minutes for welding one joint.
4. Pelican Engine coupled with metropolitan welding set costing about Rs 9,000 and Nardbur Engine coupled with Quosiare generator costing Rs 6,000 were used for welding.
5. Voltage adopted was 30 on load and 90 without load.
6. Current adopted was 200 amps with 8 gauge and 180 amps with 10 gauge.
7. Each weld cost Rs 5.75.

Appendix 5

CONCRETING

1. The total quantity of concrete was 5700 cubic feet in each span of 71 feet comprising of :
   (a) 3700 cubic feet in 4 T–beams.
   (b) 1700 cubic feet in deck slab.
   (c) 300 cubic feet in cross beams and kerbs.
2. The cost was Rs 33,000.
3. Concreting of the whole span was done in $3\frac{1}{2}$ days—2 days for beams and $1\frac{1}{2}$ days for deck slabs.
4. Maximum quantity of concreting that could be done per day was 17 units.
5. Two Millar's mixers were used.
6. The labour charges for 100 cubic feet of concrete worked out to Rs 20.

Appendix 6

DESIGN OF PENNAR BRIDGE

Intermediate Beams:

Class '*AA*' single lane

Track load $70/2 = 35$ tons.

Impact 10 per cent $= 1.1 \times 35 = 38.5$ ton.

Maximum Reaction on $B = 38.5\left(1+\frac{0.25}{7}\right) = 39.88$ tons.

Maximum B.M. due to L.L. $= \frac{39.88}{2}(37-3) = 677.9$ ft-tons.

D.L. B.M. $= 855.6$ ft-tons (i.e.) $W = 2800$ lb per rft.

$\therefore$ Total B.M. $= 1534$ ft-tons.

End Beams:

Foot path load $85 \times 5 = 425$

Sand filling $= 260$

$\overline{685}$ lb per r ft.

Wheel loads class *A*.

Reaction on the end beam.

0.9048 W (1.16)

Reaction due to front wheel $= \frac{3000 \times .9048 \times 1.16}{2240} = 1.406$ tons

Do. rear wheel $= \frac{12500 \times .9048 \times 1.16}{2240}$

$= 5.856$ tons.

Do. trailer wheel $= \frac{7500 \times .9048 \times 1.16}{2240} = 3.514$ tons.

$$R = \frac{28.58 \times 35.92}{74} = 13.872$$

Maximum B.M. due to L.L. $= 13.87 \times 35.92 - 3.514\,(30+20+10)$

$= 287.4$ ft-tons

D.L. B.M. $= 1213$ ft-tons (i.e.) $W = 3970$ lb per sq. ft.

$\therefore$ Total $= 1213 + 287.4 = 1500$ ft-tons.

$\therefore$ Design all beams for 1534 ft-tons.

$\therefore$ As per this load distribution the end beams take 287 ft. tons and the interior beams take 678 ft-tons of the live load (i.e.) 1 : 2.35 ratio.

The calculated deflections $= 0.24$ for the interior beam

,, ,, $= 0.10$ for the exterior beam

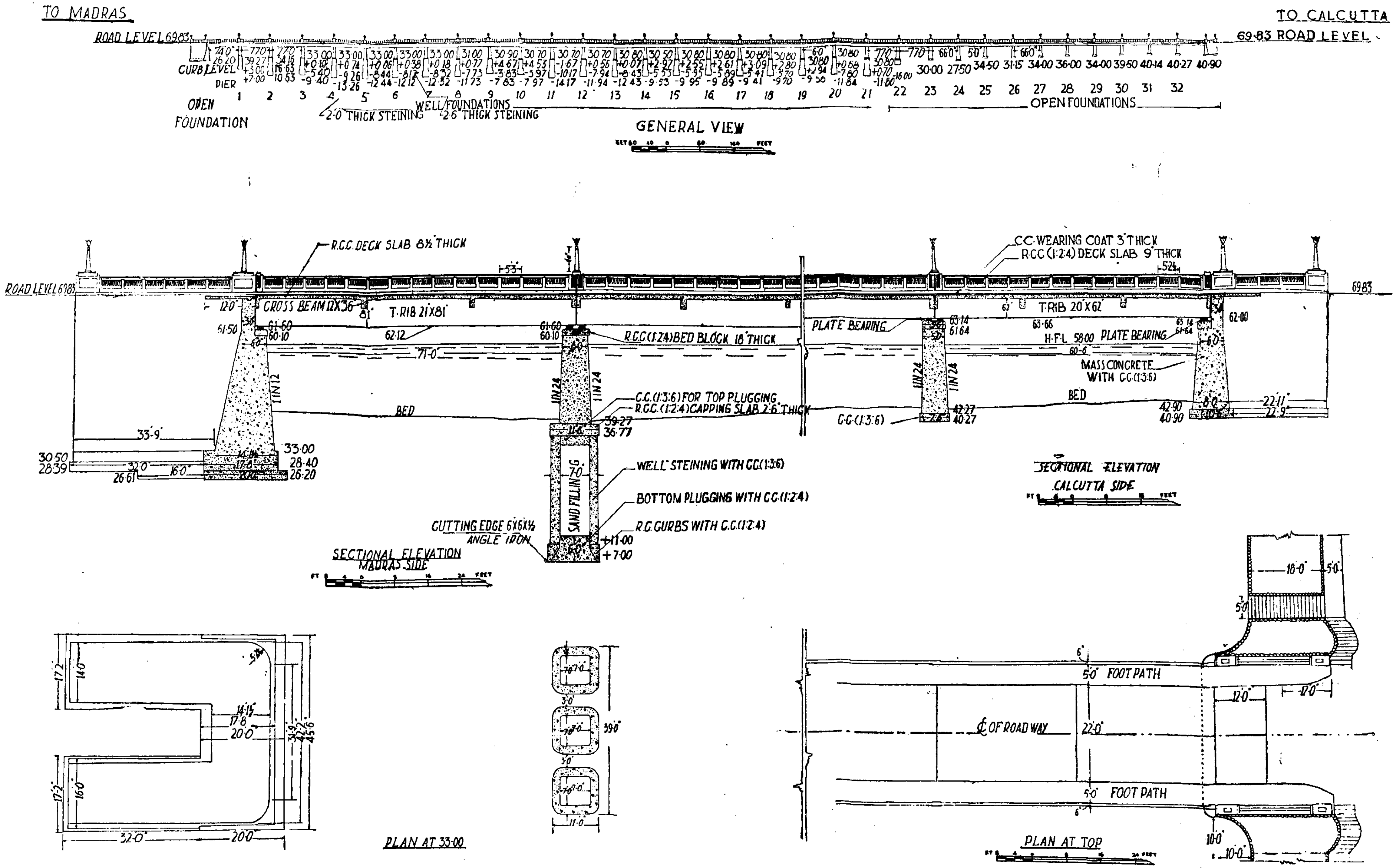
Actual observed deflection.

Interior $= 0.11$ and exterior $= 0.1$

(i. e.) $2 \times .11 + 2 \times .1 = 0.42$ total against calculated deflection $2 \times .24 + 2 \times .1 = 0.68$.

$\therefore$ The observed deflection $= 42/68 = 62$ per cent of the calculated equivalent design loads for test purpose.

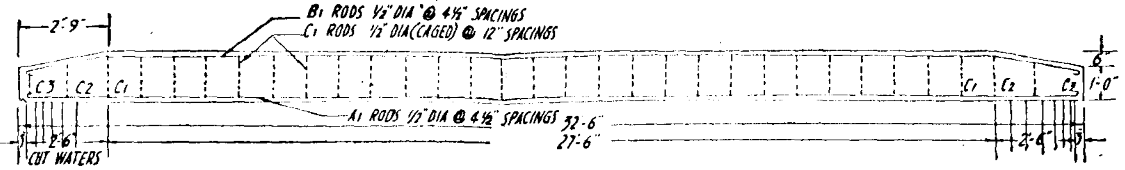
PLATE I



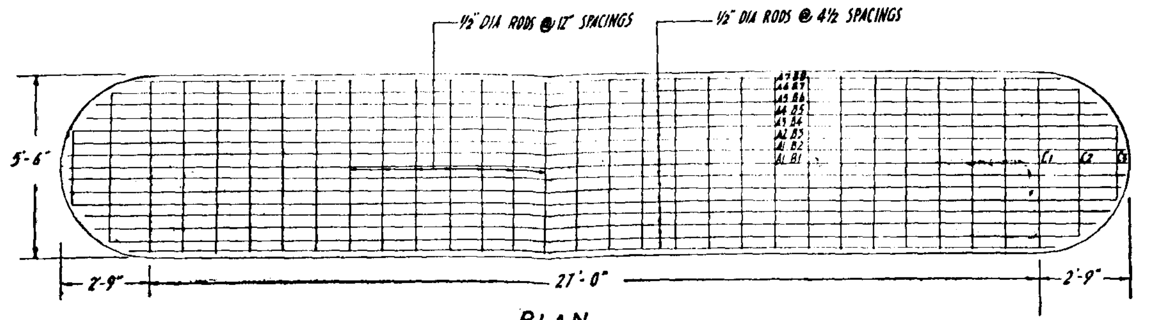
LONGITUDINAL SECTION PENNAR BRIDGE

PLATE II

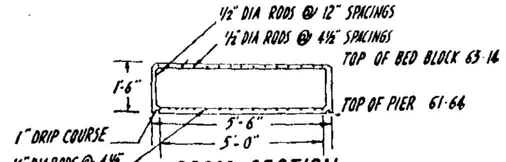
BED BLOCK FOR 5'-0" WIDE PIER



LONGITUDINAL SECTION



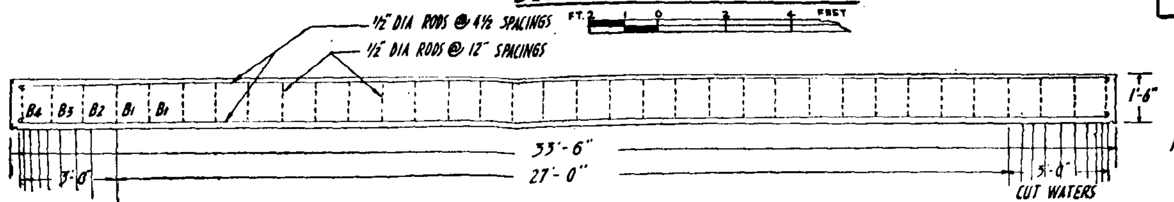
PLAN



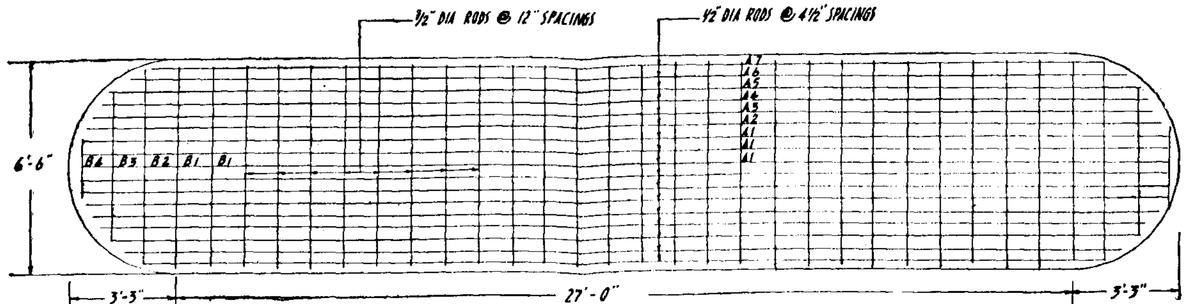
CROSS SECTION

S.No.	NAME	DIA	LENGTH	NOS.	TOTEL LENGTH	WEIGHT
1	A 1	1/2"	32'-9"	3	98'-3"	
2	A 2	1/2"	32'-6"	2	65'-0"	
3	A 3	1/2"	32'-4"	2	64'-8"	
4	A 4	1/2"	31'-9"	2	63'-6"	
5	A 5	1/2"	31'-8"	2	62'-6"	
6	A 6	1/2"	30'-3"	2	60'-6"	
7	A 7	1/2"	28'-5"	2	56'-10"	
8	B 1	1/2"	33'-0"	1	33'-0"	
9	B 2	1/2"	32'-10"	2	65'-8"	
10	B 3	1/2"	32'-7"	2	65'-2"	
11	B 4	1/2"	32'-5"	2	64'-10"	
12	B 5	1/2"	31'-11"	2	63'-10"	
13	B 6	1/2"	31'-4"	2	62'-8"	
14	B 7	1/2"	30'-4"	2	60'-8"	
15	B 8	1/2"	28'-6"	2	57'-0"	
16	C 1	1/2"	14'-2"	28	396'-8"	
17	C 2	1/2"	12'-3"	2	24'-6"	
18	C 3	1/2"	6'-4	2	12'-8"	
					TOTAL : 1377'-11"	921 LBS.
					ADD FOR WASTAGE	46 "
					BINDING WIRE	9
						976 LBS.

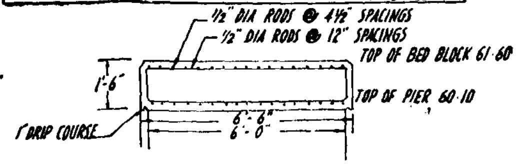
BED BLOCK FOR 6'-0" WIDE PIER.



LONGITUDINAL SECTION

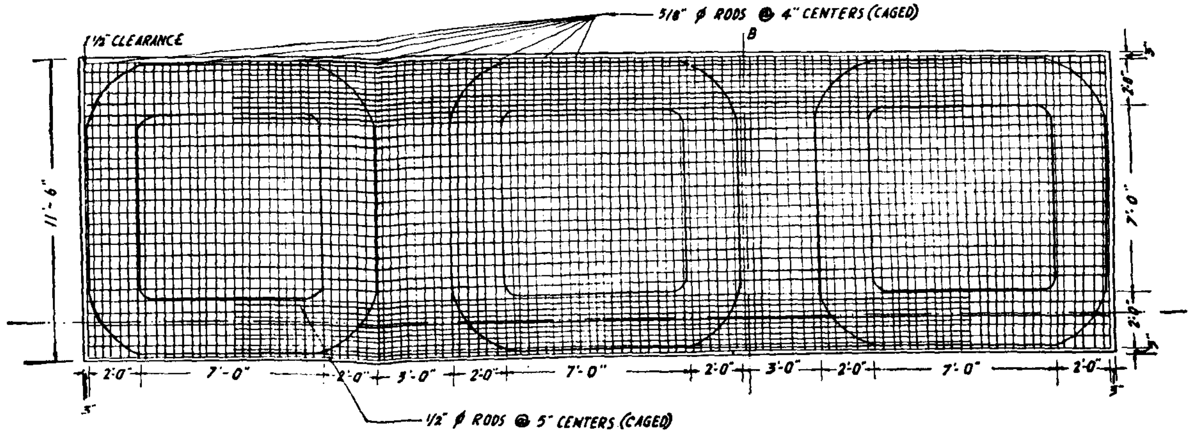


PLAN

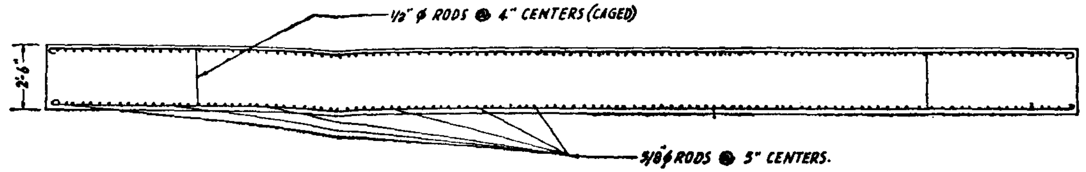


CROSS SECTION

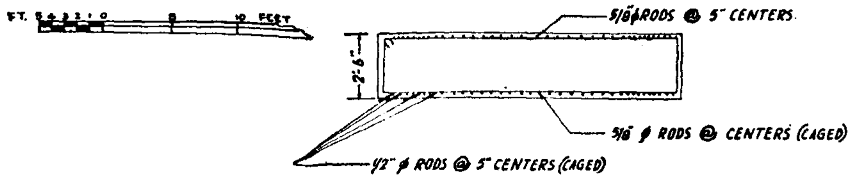
S.No.	NAME	DIA	LENGTH	NOS	TOTAL LENGTH	WEIGHT.
1	A 1	1/2"	33'-9"	10	337'-6"	
2	A 2	1/2"	33'-3"	4	133'-0"	
3	A 3	1/2"	33'-0"	4	132'-0"	
4	A 4	1/2"	32'-3"	4	129'-0"	
5	A 5	1/2"	31'-7"	4	126'-4"	
6	A 6	1/2"	30'-3"	4	122'-0"	
7	A 7	1/2"	30'-0"	4	120'-0"	
8	B 1	1/2"	16'-2"	28	452'-8"	
9	B 2	1/2"	15'-4"	2	30'-8"	
10	B 3	1/2"	13'-2"	2	26'-4"	
11	B 4	1/2"	7'-8"	2	15'-4"	
12.	SUPPORT RODS FOR KEEPING BOTTOM TOP RODS IN POSITION IN THE MIDDLE	1/2"	2'-3"	14	31'-6"	
					TOTAL : 1656'-4"	1106 LBS
					ADD FOR WASTAGE	55 "
					BINDING WIRE	10
						1171 LBS.



PLAN



SECTION @ AA

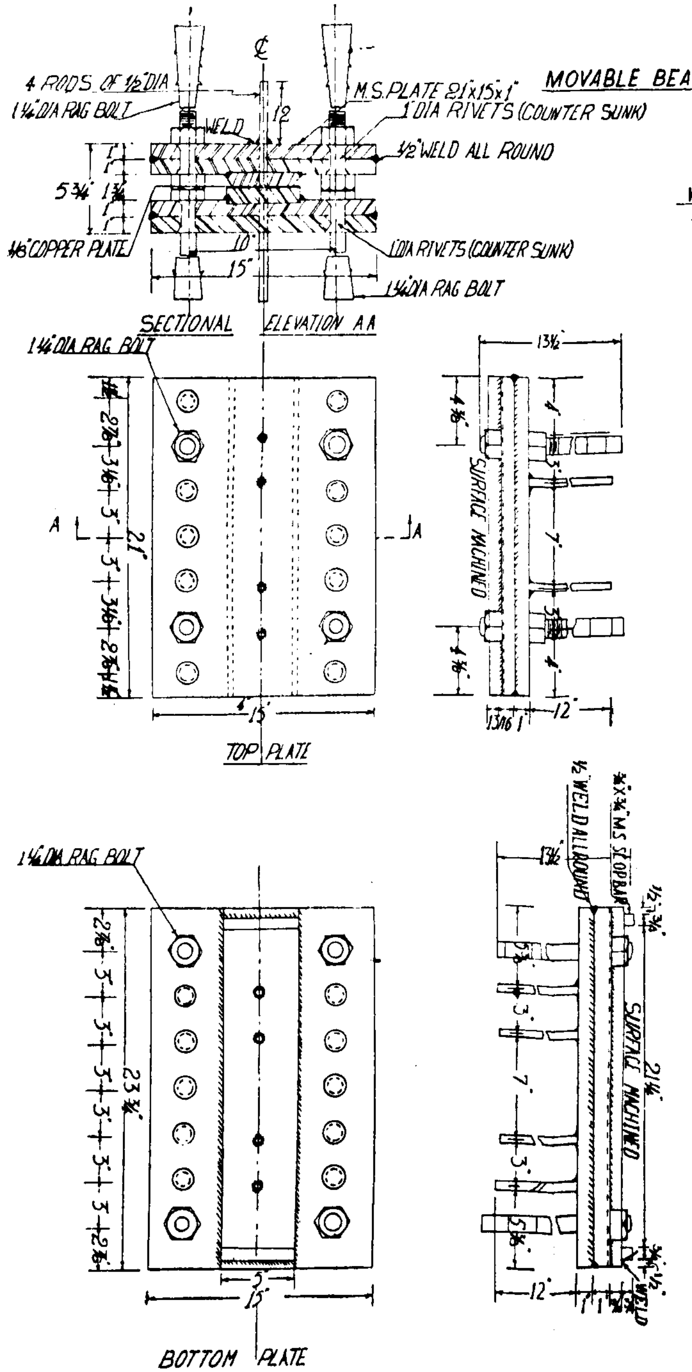


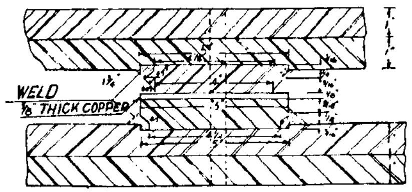
SECTION @ BB

S.No.	DESCRIPTION.	LENGTH INCLUDING OVERLAPING	NUMBERS	TOTAL LENGTH	TOTAL WEIGHT
	5/8" ϕ RODS				
1	39'-3"	40'-0"	2x28	2240	
2	11'-3"	28'-0"	119	3332	
				5572	5811.6
3.	1/2" ϕ RODS				
	20'-0"	66'-0"	10	660	440 9
					6252.5 LBS
		BINDING WIRE @ 1 LB PER 112 LBS OF STEEL			55.8 -
					6308.3 -

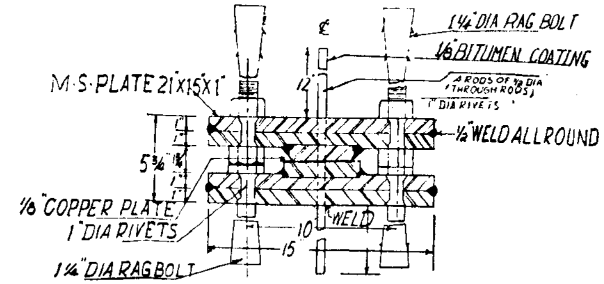
DETAILS SHOWING REINFORCEMENTS FOR BED BLOCK AND CAPPING SLAB PENNAR BRIDGE

PLATE III



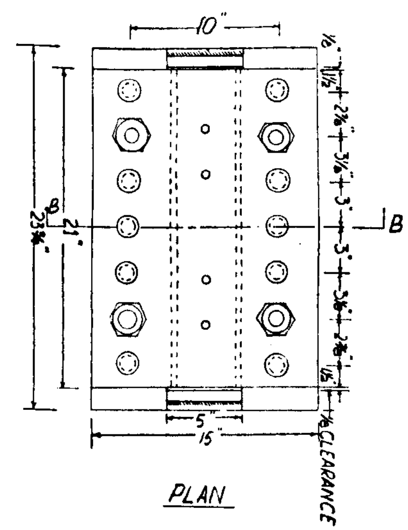


DETAILS OF CONTACT PLATE



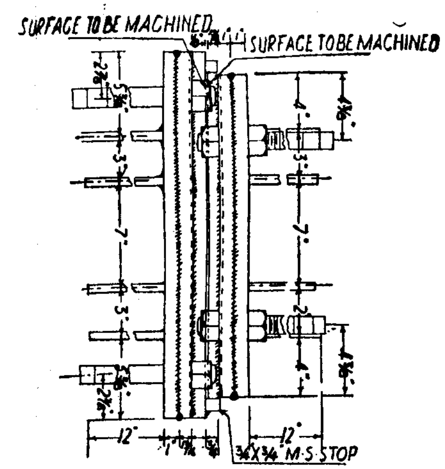
SCHEDULE OF REQUIREMENTS

ITEM	DESCRIPION	NUMBER	REMARKS	MATERIAL
1	TOP PLATE 2NOS 21"X15"X1"	264	TWO PLATES TO BE JOINED BY RIVETING & WELDING AS SHOWN	MILD STEEL
2	BOTTOM PLATE 2NOS 23¾"X15"X1"	264	TWO PLATES TO BE JOINED BY RIVETING & WELDING AS SHOWN	"
3	CONTACT PLATE 21"X5"X1"	264	TO BE WELDED TO TOP PLATE COMPLETE AS SHOWN	"
4	CONTACT PLATE 23¾"X5"X1"	264	TO BE WELDED TO BOTTOM PLATE COMPLETE AS SHOWN	"
5	STOP BAR 5"X¾"X¾"	528	TO BE WELDED TO BOTTOM PLATE COMPLETE AS SHOWN	"
6	ANCHOR RODS ½" Φ 14½" LONG	1056	TO BE WELDED TO TOP & BOTTOM PLATE AS SHOWN	"
7	THROUGH RODS ½" Φ 30" LONG	528	TO BE WELDED TO BOTTOM PLATE AS SHOWN	"



PLAN

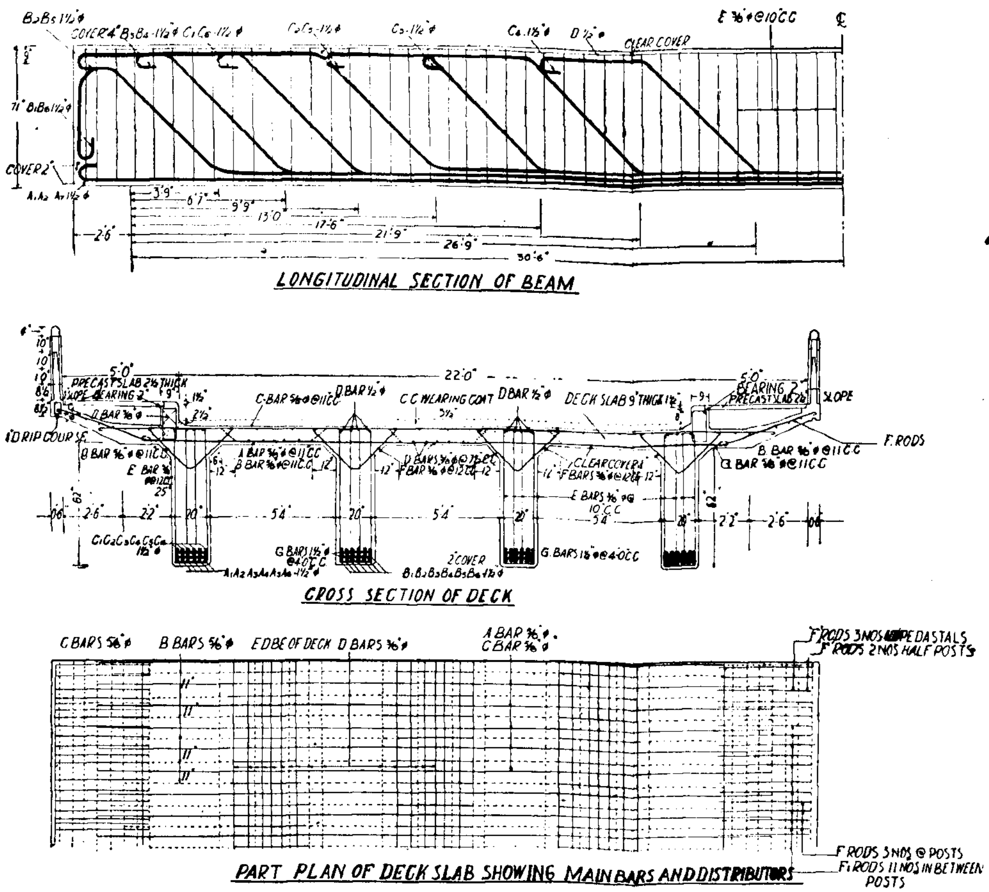
NOTE:- THE DETAILS OF BEARINGS SHOWN HOLD GOOD FOR BOTH 71' & 61' SPANS



END VIEW

M. S. PLATE BEARINGS
PENNAR BRIDGE

PLATE IV



NAME	DIA IN	LENGTH	NOS	TOTAL LENG FT	WEIGHT IN LBS	SHAPE OF BAR
A1-A6	1½"	67'-8"	24	1624		
B1 B6	1½"	78'-0"	8	624		
B2 B5	1½"	71'-8"	8	573		
B3 B4	1½"	66'-2"	8	529	27114	
C1 C6	1½"	59'-0"	8	472		
C2 C5	1½"	50'-0"	8	400		
C3	1½"	41'-4"	4	165		
C4	1½"	31'-4"	4	125		
D	½"	66'-3"	16	1060		
E	½"	27'-4"	180	5020	4441	L=2(5'-7"+10")+10"
E1	½"	1'-5"	88	125		(TO SUPPORT CRANKED)
F	⅜"	5'-4"	268	1429	536	
G	1½"	1'-4"	92	123	739	HAUNCH
H	⅞"	66'-0"	8	568	214	HANGER RODS

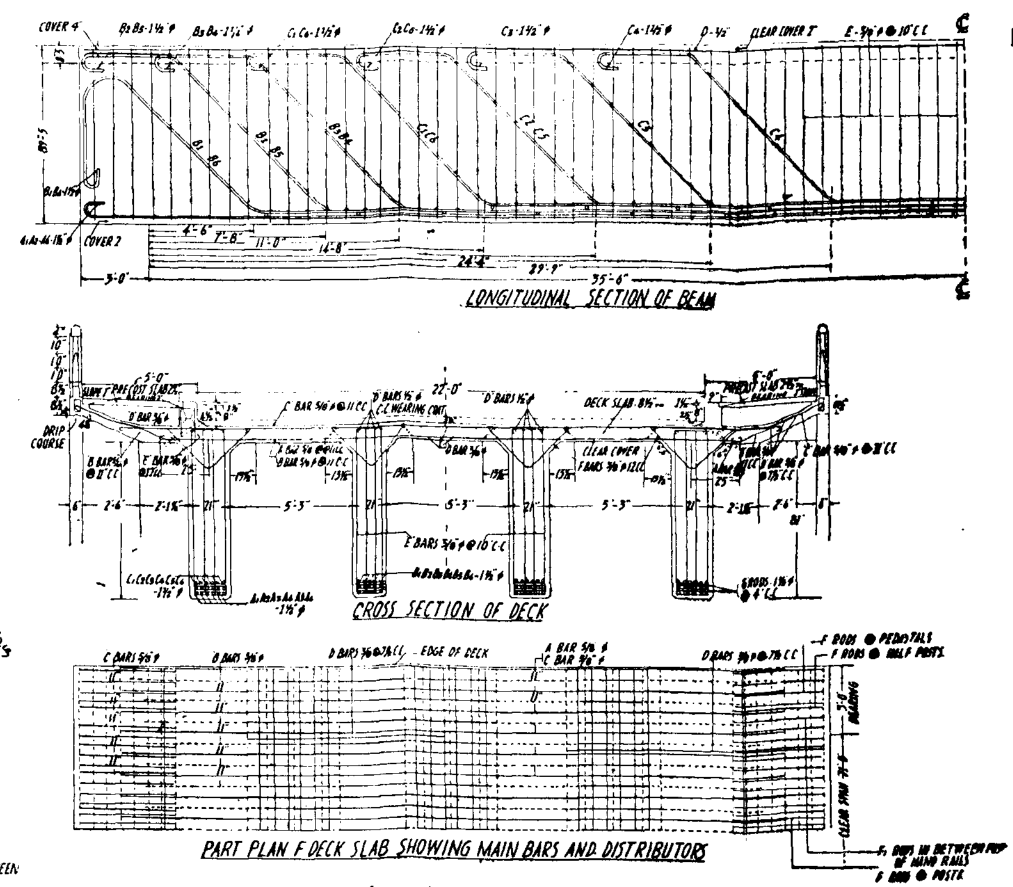
33144
ADD 5% FOR SPLICING & WASTAGE 1657
FOR BAIDING WIRE 290
TOTAL 35091

NAME	DIA IN	LENGTH	NOS	TOTAL LENGTH	WEIGHT	SHAPE OF BAR
Q	⅝"	26'-2"	71	1258		
B	⅝"	29'-3"	72	2166	6319	
C	⅝"	29'-1"	73	2094		
D	⅝"	66'-3"	42	2805		
D1	½"	66'-5"	27	1793		
E	⅜"	4'-8"	134	603	3250	L=2(7)4"+10
F	½"	5'-0"	98	490		
F1	⅜"	5'-0"	264	1320		IN CANTILEVER PORTION

9569
ADD 5% FOR WASTAGE & SPLICING 478
BINDING WIRE 86
TOTAL 10133 LBS

FT. 0 1 2 4 6

REINFORCEMENT DETAILS FOR DECK CLEAR SPAN 61 FT. 0 IN.



REINFORCEMENT FOR T BEAM (ONE SPAN)

NAME	DIA IN	LENGTH FT IN	NOS	TOTAL LENGTH FT	WEIGHT LBS	SHAPE OF BAR
A1 A2-A6	1½	78-10	24	1892	11378	
B1-B6	1½	91-6	8	732	4360	
B2 B5	1½	84-2	8	673	4049	
B3 B4	1½	77-10	8	623	3743	
C1 C6	1½	69-8	8	558	3352	
C2 C5	1½	60-0	8	481	2890	
C3	1½	50-0	4	201	1214	
C4	1½	39-6	4	158	949	
D	½	77-5	16	1239	828	
E	½	34-4	216	7416	5042	2x16'-11"+33'-10"
E1	½	1-6	88	132		(TO SUPPORT CRANKED RODS)
F	⅜	5-5	312	1690	635	1'-9"
G	1½	1-5	108	153	919	
H	⅝	77-4	8	619	233	HAUNCH HANGER RODS

39592
ADD 5% FOR SPLICING ETC 1979
BINDING WIRE 353
41926 LBS

NOTE :-
INNER RADIUS OF BENDS SMALL NOT BE LESS THAN 8" DIAMETERS

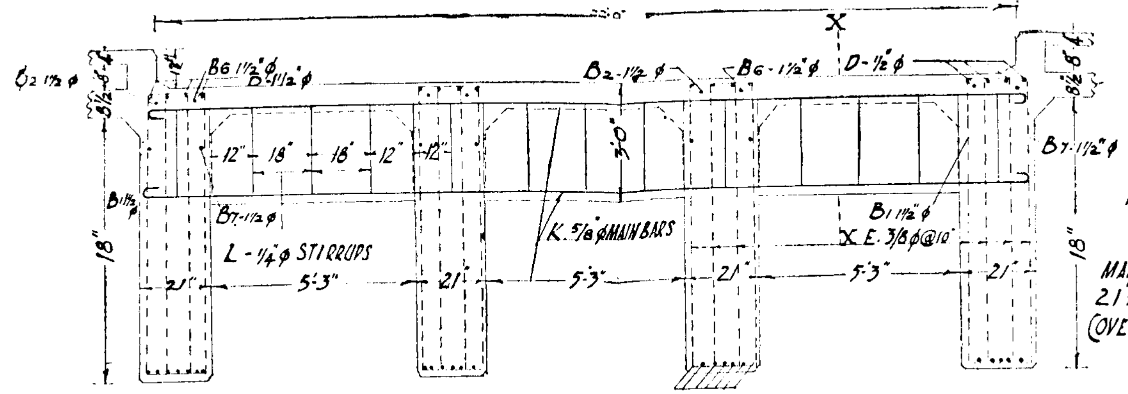
REINFORCEMENT FOR SLAB (ONE SPAN)

NAME	DIA IN	LENGTH FT IN	NOS	TOTAL LENGTH FT	WEIGHT LBS	SHAPE OF BAR
A	⅝"	26'-2"	85	2172		
B	⅝"	29'-5"	84	2457	7396	
C	⅝"	29'-1"	85	2472		
D	⅝"	77'-5"	42	3244		
D1	½"	77'-5"	27	2090		
E	⅜"	4'-6"	156	702	3785	
F	½"	5'-0"	98	490		
F1	⅜"	5'-0	308	1540		

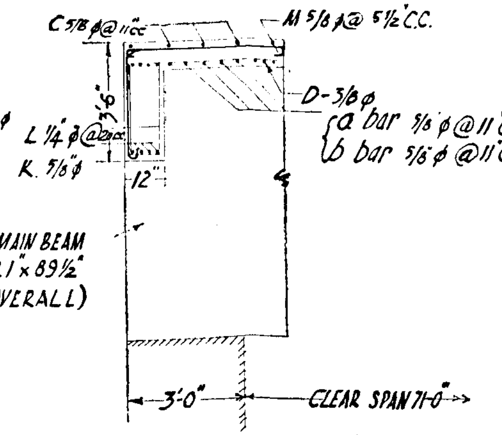
11181
ADD 5% FOR SPLICINS ETC 555
BINDING WIRE 98
11834

REINFORCEMENT DETAILS FOR DECK CLEAR SPAN 71 FT. 0 IN. PENNAR BRIDGE

PLATE V



LONGITUDINAL SECTION OF CROSS BEAM (FOR 71FT T BEAM)

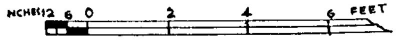


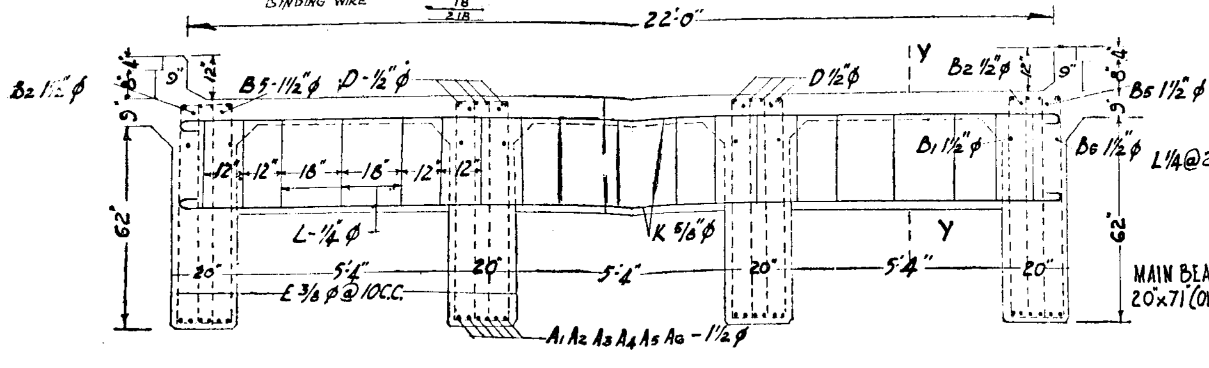
SECTION ON XX

REINFORCEMENTS FOR CROSS BEAMS IN ONE SPAN

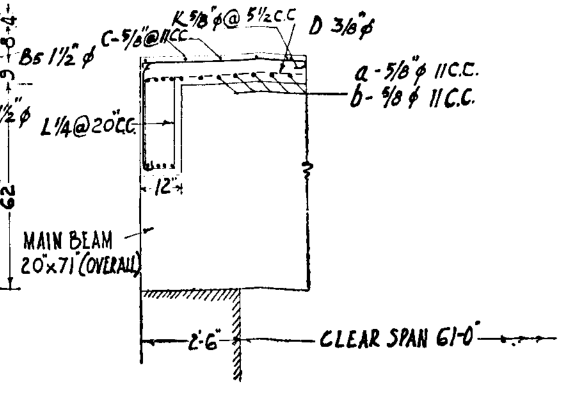
NAME	DIA	LENGTH	NOS	TOTAL LENGTH	WEIGHT LBS	SHAPE OF BAR
K	⅝"	23'-4"	32	746'-8"	779	6" 6" 22'-4"
L	¼"	6'-8"	76	253	85	2'3½" 2.3½ oo $l = 2(2.3+8")+10"$
M_1	⅝"	7'-6"	72	540	563	5'-7" 2'3"
M_2	⅝"	8'-6"	72	612	638	6" 6" 7'-6"

ADD FOR WASTGE & SPLICING 3% 43
BINDING WIRE 7
TOTAL 2065
ADD FOR WASTAGE 5% 103
BINDING WIRE 18
218





LONGIGITUDINAL SECTION OF CROSS BEAM (FOR 61 FT BEAM)

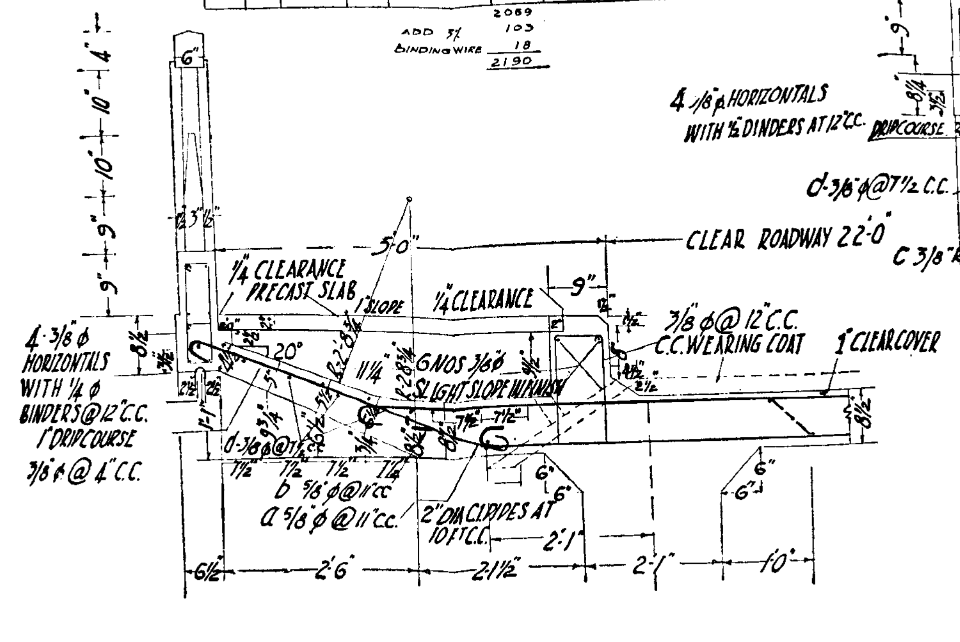


SECTION ON YY

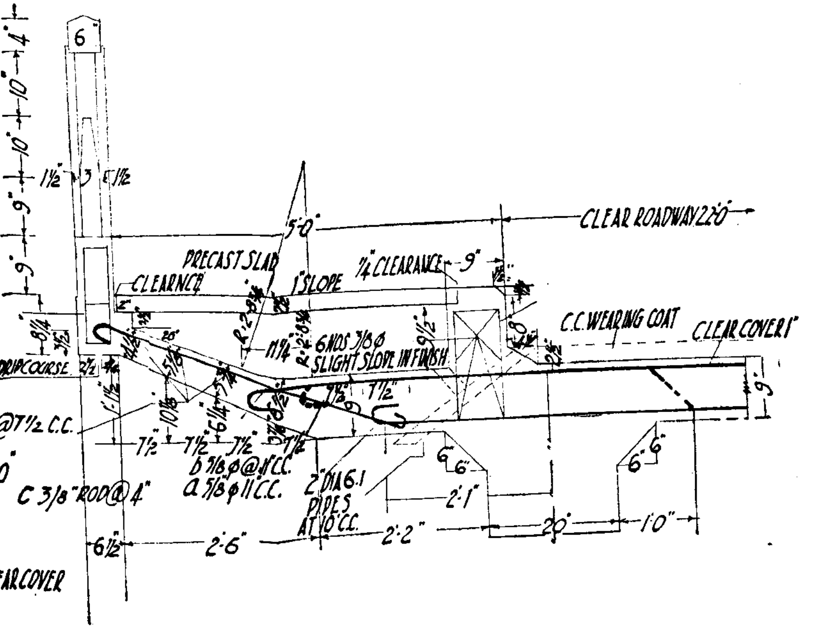
REINFORCEMENT FOR CROSS BEAMS IN ONE SPAN.

NAME	DIA	LENGTH	NOS	TOTAL LENGTH	WEIGHT LBS	SHAPE OF BAR
K	⅝"	23'-2"	32	741	785	6" 6" 22'-2"
L	¼"	6'-8"	76	506	85	2'3½" oo $l = 2(2.3+8)+10$
M1	⅝"	7'-6"	72	540	563	5'-7" 6"
M2	⅝"	8'-6"	72	612	638	6" 6" 7'-6"

2059
ADD 5% 103
BINDING WIRE 18
2190



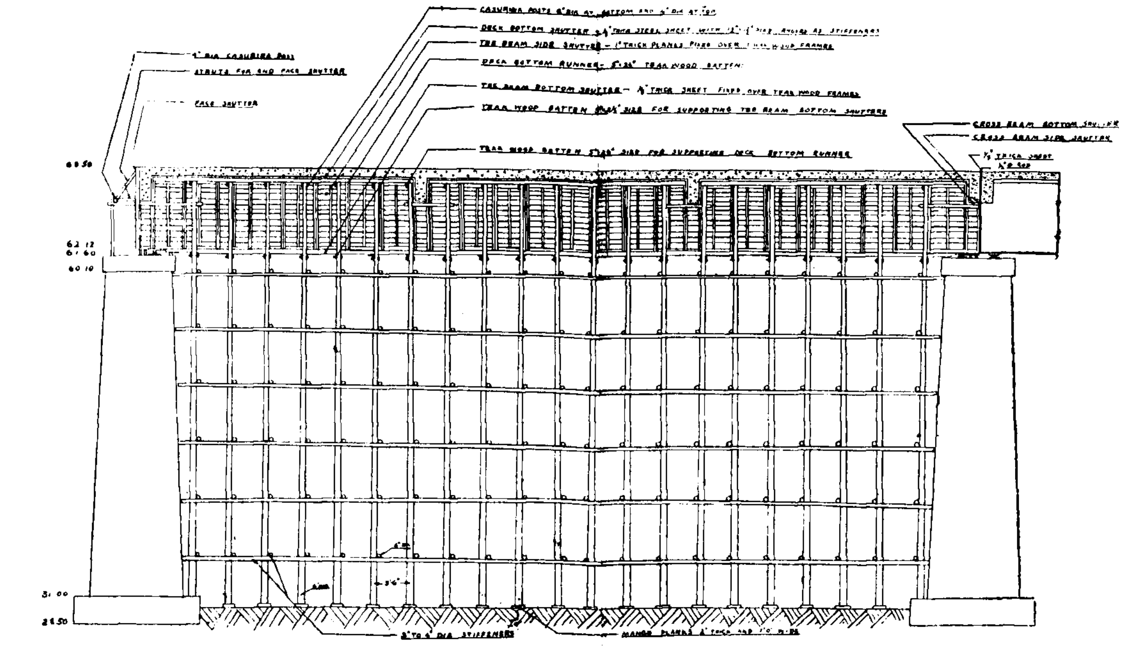
DETAILS OF FOOT PATH FOR 71 FT SPAN DECK



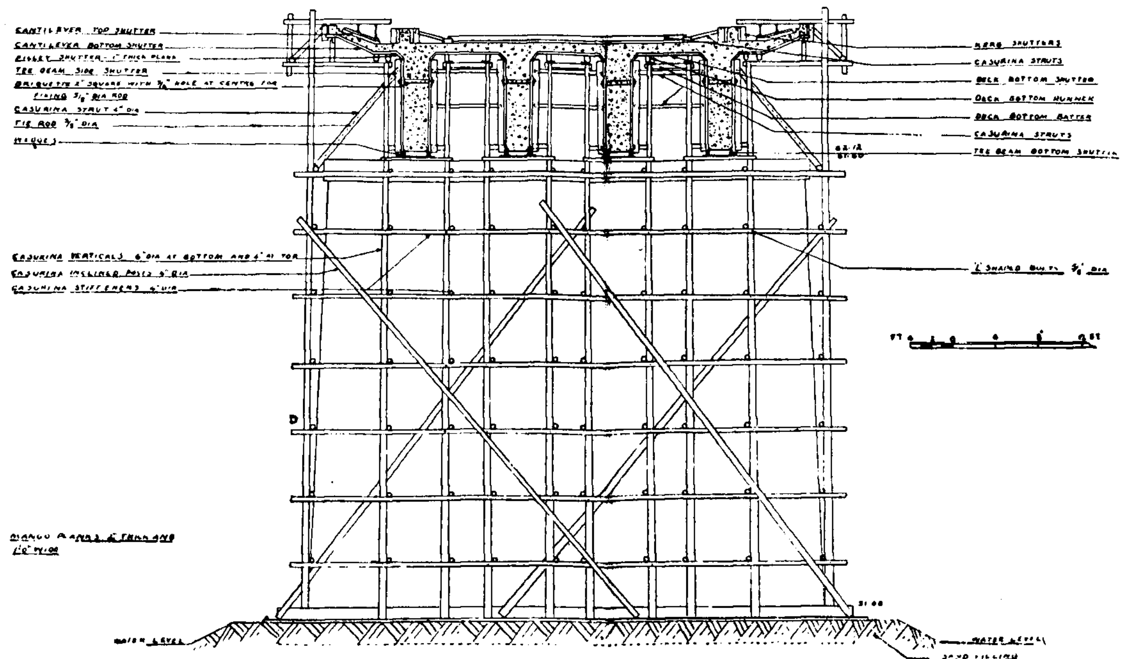
DETAILS OF FOOT PATH FOR 61'-0" SPAN DECK

DETAILS OF CROSS BEAMS AND FOOTPATH PENNAR BRIDGE

PLATE VI



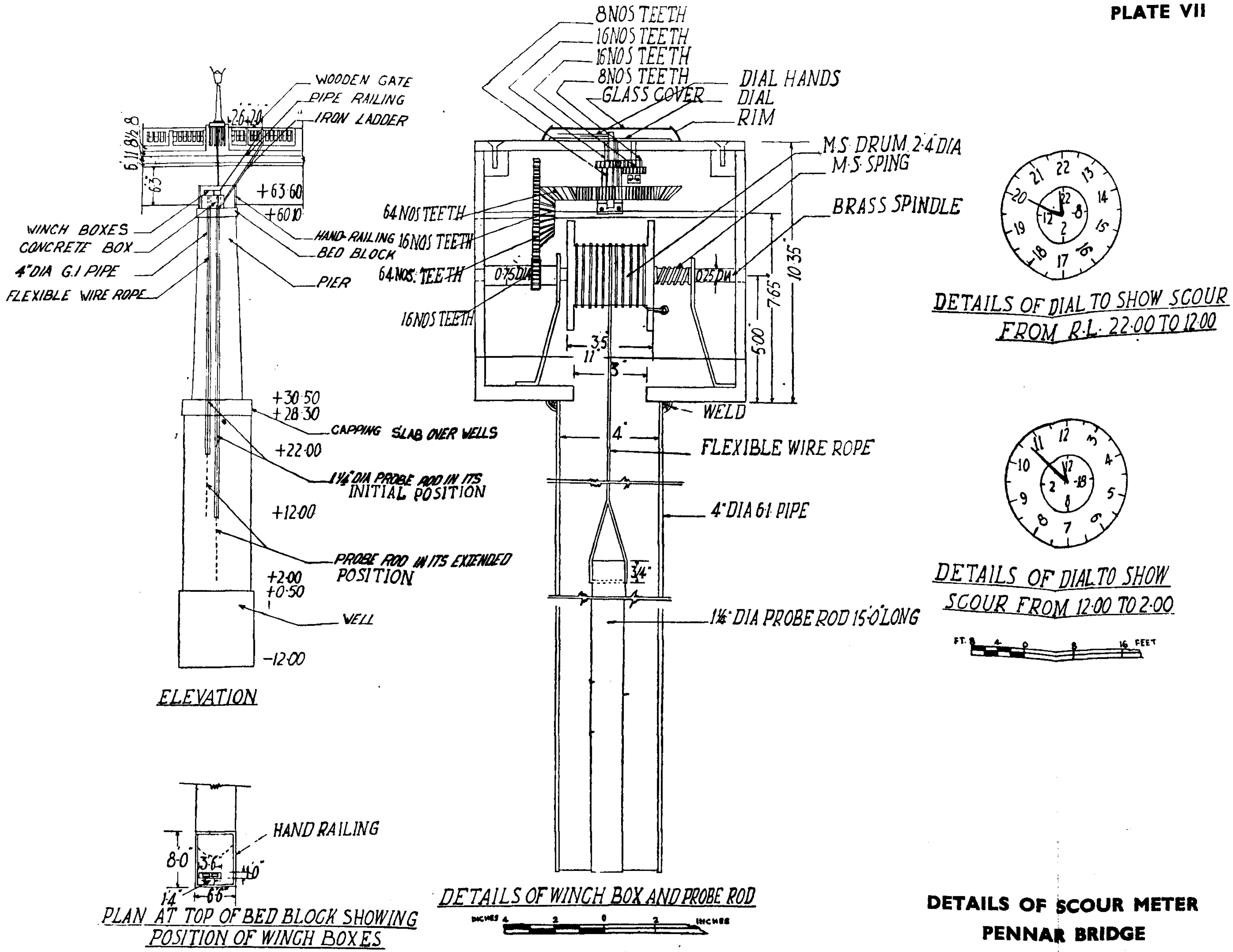
LONGITUDINAL SECTION



CROSS SECTION

DETAILS OF SCAFFOLDING AND CENTERING PENNAR BRIDGE

PLATE VII



DETAILS OF SCOUR METER PENNAR BRIDGE

1. SURFACE DRESSING WITH R. T. 4
2. PRESTRESSED CONCRETE PAVEMENTS

"SURFACE DRESSING WITH R. T. 4"

By

K. C. De*

CONTENTS

INTRODUCTION

The present position of our country needs a quick run in the development of every section of activity in order to reach the level of other countries. For this, technical study and research work in relation to the resources and the climatic conditions of our country will no doubt play an important role. In many fields, so long, we were much behind the progress of other countries and were following their results. We have now to do our own experimentation and research and evolve methods and techniques to suit our conditions.

SELECTION OF GRADE OF TAR

Among bituminous road surfacing materials, Road Tars manufactured out of crude tar, obtained as a by-product mainly in the preparation of coke from the coal of our country is one of the indegenous items. In manufacturing different grades of Road Tars, we followed for a long time the specifications of countries whose climatic conditions were different from ours. But afterwards, in actual working with the materials, it became obvious that the grade of Road Tar suitable for a particular

* Roads Engineer, M/s. Bengal Chemical & Pharmaceutical Works Ltd., Calcutta.

type of work in cold countries may not be suitable for the same type of work in a country like ours, where variation of atmospheric temperature is wide and the upper limit is much higher than that of the cold countries. Besides temperature, humidity and rainfall are also to be taken into consideration in the slection of grade of Road Tars for the different types of works in our country. Low viscosity Road Tar may produce good result in a cold country, but in our hot climate it has got many disadvantages in respect of laying and the life of the roads.

The highly fluid nature of Road Tar has got an advantage, since it ensures a reasonable penetration of the macadam and also allows the blinding material to sink easily into it. The fluidity of Road Tars varies with the temperature and the grade selected for the work. Road Tar of lower viscosity requires much less temperature than Road Tar of higher viscocity to bring it to the working fluid condition to obtain the above mentioned advantages and the same wetting quality. In surface dressing, when heated Road Tar is spread on to the road, its viscosity remains low, but in contact with the road surface, it rapidly cools to road temperature and the viscosity gradually rises. The blinding materials that have been rolled in to some extent, just after spreading over the painted surface, should then be gradually wetted by the Road Tar under traffic pressure in relation to the road temperature and the grade of Road Tar used, in such a way that the blinding materials do not get disturbed either by the suction or the impact of the traffic wheel, but will be compacted by the kneading action of the traffic, so that the particles become more closely interlocked and the binder becomes less exposed to the weather. In this way under ideal conditions, if adequate quantity is used, Road Tar should penetrate into or revive the surface over which it is spread to the extent required for the purpose and come up just to cover the vertical sides of the blinding materials with minimum thickness and not the top surface of it. Thus a close mosaic surface will be obtained to give the longest life to the road with good riding quality.

It is now generally recognised that selection of grade of Road Tar is one of the major factors for obtaining long life out of surface dressing which is an economic type of treatment to be received by 80 per cent of the roads of our country. The selection of the grade should be in conformity with the temperature the road surface can attain; otherwise, if Road Tar of lower viscosity is used in the plains where road surface temperature is quite high enough to keep the viscosity too low to offer sufficient resistance to the displacing action of the traffic in both newly applied initial or subsequent coats of dressing, the blinding materials will roll over under traffic exposing their tarry surface to the next traffic, which then will pick them up and throw them toward the sides. On the other hand, the wet tar on the wheels will go on dislodging further the blinding materials causing quick deterioration in the surface texture as well as of water-proofing condition in case of application of initial coat over water-bound course. The adhesion of the chippings is a slow process. They take up their final positions and bed themselves in slowly and they must be given reasonable time to get their right place. It is not fair to tear them out by the traffic wheels wetted with Tar which, due to lower viscosity, quickly comes on the top as the blinding materials are squeezed in under traffic, before they have had time to fully settle in.

There has now been considerable experience in surface dressing and it is seen that when stone chips, which we generally use as blinding material, are spread over the road surface painted with heated Road Tar and adequate rolling is done without crushing the chips Road Tar comes up to cover the vertical sides of the chips, upto a certain height. Road Tar as binder then should gradually work up under traffic pressure, in such a way, that the road temperature will not be able to bring down its viscosity so low that it cannot offer sufficient resistance to the displacing action of the traffic. The road temperature will just be able to soften the Road Tar to help in working up to lend more and more lateral support to the individual chippings. In practical working it has been observed that the wetting quality of high viscosity Road Tar (R. T. 4) at the road temperature of India is quite satisfactory and it also can bind the top layer of the original surface with the blinding materials used in surfacing and can remain in plastic solid condition with age in better ways than the low viscosity grades.

EXPERIMENTAL WORK

Photo 1 is a laboratory test track designed by the author. By the help of this apparatus nature of working of Road Tars of both higher and lower viscosity as binders at the road temperature of the working season of plains in West Bengal has been tested. It is found that the result with High Viscosity Road Tar (R. T. 4) is quite satisfactory and approaches the theoretical conception. The Test Track has been made to run the road under two 2 in. flat iron wheels having almost 6 mds. load on each wheel.

Photo 2 shows the surface condition of the section, of the road on the above Test Track laid with R. T. 4 and R. T. 3, using the same quantity of binder in each case, after one lac passes of wheels on the surface under the above mentioned load. In the case of R. T. 3 the surface bled and dry sand had to be spread on the surface to save the wheel face from coming in contact with the bled binder. The binder, being squeezed, went out at the sides as the interlocking of the stone chips increased under traffic. The surface with R. T. 4 under the same conditions looked to be mosaic in character with binder worked up, upto the surface covering the vertical sides of the chips with minimum thickness. To find out whether use of less quantity of Road Tar of grade R. T. 3 can save the situation of bleeding, a section was laid on the Laboratory test track with R. T. 3 at the rate of 30 lb per sq. 100 ft. over water-bound surface, blinding with 4.5 cu. ft. of ½ in. stone chips. The result was that at road temperature (108°F) R. T. 3 became thin to the extent that the chips easily moved and pressed the binder to the sides. In this way accumulation of binder occurred in places and bleeding took place there. Due to lubricating action of thin binder the chips are disturbed under traffic. This does not happen in the case of High Viscosity Road Tar (R. T. 4).

From the results of the laboratory test track the working specification for initial coat of surface dressing with R. T. 4 using ½ in. dry stone chips as blinding material, may be suggested as under :

Type of Water Bound Macadam			
Dense & Compacted		Open Texture & Porous	
Road Tar per 100 sq. ft	Chips per 100 sq. ft	Road Tar per 100 sq. ft	Chips per 100 sq. ft
40–45 lb	4.5–5 cu. ft.	45–50 lb	4.5–5 cu. ft.

In the plains of India the general practice is to use R. T. 2 & R. T. 3 in initial and subsequent coats of surface dressing respectively, but in consideration of facts stated above, Road Tar of still higher viscosity i. e., grade R. T. 4 should be used for successful surface dressing. In seasons when generally surfacing work in the plains is carried out, the accumulated road temparature varies from 80°F to 120°F. The mean equiviscous temparature (E. V. T.) of Road Tars of Grades R. T. 3 and R. T. 4 are 44°C and 55°C respectively. When converted to Fahrenheit they come approximately to 110°F and

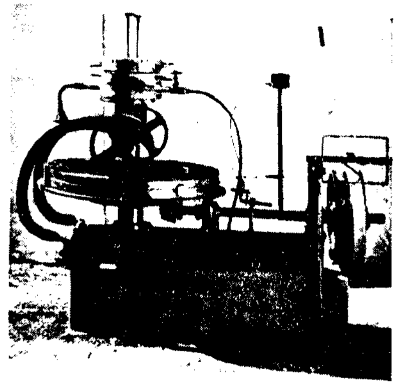


Photo 1.

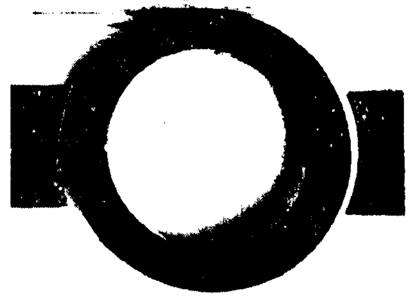


Photo 2.

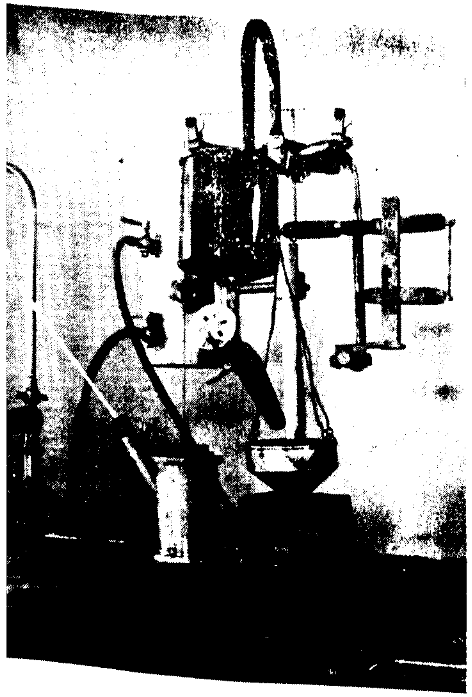


Photo 3.

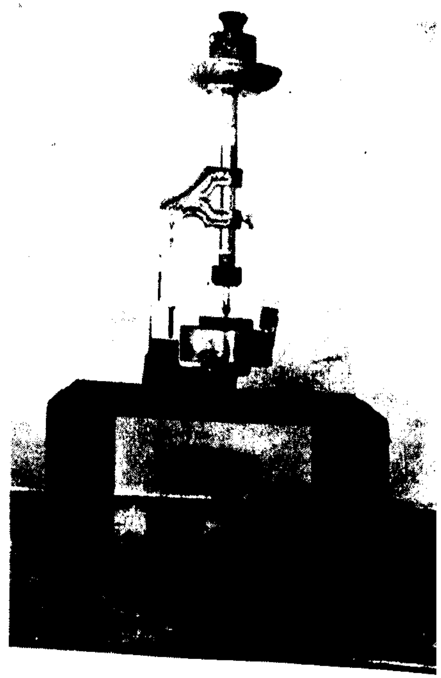


Photo 4.

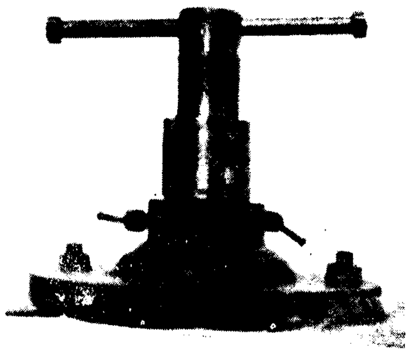
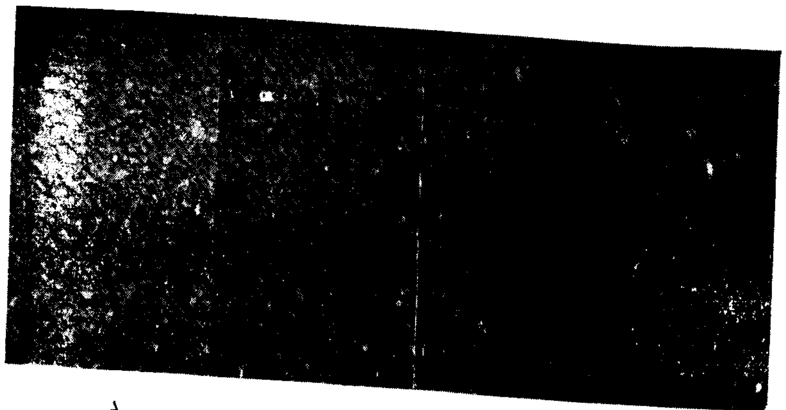


Photo 5



A B C D

Photo 6

130°F. This difference of 20°F will count to keep the viscosity of R. T. 4 at road temperature in such a condition that the chips under traffic will gradually dip into the Road Tar, but will be firmly held right from the start. The initial adhesive strength of R. T. 4 at road temperature has been found by a special apparatus designed by the Author, Photo 3, to be almost double that of R. T. 3 at the same temperature.

In carrying out this experiment 0.1 gram of binder is taken on a metal seat of ½ sq. inch in area and put on a levelled base in a hot air oven under constant temperature for half an hour for even distribution of the binder on the seat. Another piece of metal of exactly the same area is then put to cover the painted surface with 100 grams load on it and kept for half an hour on a levelled base in open air at room temperature and then the seat is screwed to a cylinder, through which water of any temperature can run touching the under base of the seat. In this way the temperature of the binder is controlled. Water at temperature at which the experiment is to be done is run for an hour through the cylinder. The upper metal piece is then connected with a fine flexible wire rope running over two pulleys and the other end is fixed to a weighing pan. Lead shorts are then put on the weighing pan through a special device with uniform speed till the upper metal piece comes out of the seat. Weights of the lead shorts are then taken and adhesive strength is then found out in case of different grades of Road Tar under same conditions in terms of weight in grams. In this apparatus, both vertical pull and pulls at different angles can be measured.

Readings given in Table 1 are the mean of three readings taken in each case at the same temperature:

TABLE 1

GRADE OF ROAD TAR	WEIGHTS NEEDED TO OVERCOME THE RESISTANCE OF BINDER
R. T. 3	3720 gram
R. T. 4	6520 gram

PHYSICAL AND CHEMICAL PROPERTIES

In respect of natural and normal ageing of Road Tar, which happens mostly for two factors, viz., the evaporation of oil and the internal changes of the materials due to atmospheric reactions, such as oxidation, crystallization etc., Road Tar of higher viscosity can give more life than Road Tar of lower viscosity. This is because the rate of evaporation of oil in high viscosity Road Tar is much less than in low viscosity Road Tar as the former contains oil of higher fractions more than the latter and the rate of evaporation of oil of higher fraction is much less than the oil of lower fractions. On the other hand, atmospheric reaction, the other factor, is also very slow with the oil of higher fractions than with the oil of lower fractions. Besides these advantages, high viscosity Road Tar can stand the road temperature of this country to attain the ideal conditions required for successful treatment of surface dressing. On the other hand, the lowest temperature expected in the plains of this country in winter is not the criterion to think about the brittleness of high viscosity Road Tar.

To find out the rate of evaporation of oil under road condition two sets of briquettes, one set with R. T. 3 and the other set with R. T 4, having all the materials and conditions constant, were made and weighed in a chemical balance. Three briquettes of each set were then kept under flow of air under constant temperature of 120°F for several weeks together and weights were taken after each week. The weight readings given in Table 2 will show that the rate of evaporation of oil in briquette made with R. T. 3 is more than the evaporation of oil in briquette made with R. T. 4.

TABLE 2

Briquette with Grade of Road Tar	Weights of Briquettes Taken at Intervals				Total Loss
	Original	1st week	2nd week	3rd week	
R. T. 3	41.7188	41.6240	41.5084	41.4850	0.2338
R. T. 4	41.6060	41.5600	41.5416	41.5294	0.0766

The viscosity of Road Tar is almost independent of the initial viscosity at the time of laying. As time goes on the viscosity rises till it maintains the plastic solid condition due to evaporation of oil and the interval change due to the atmospheric reactions, the rate of which varies in respect of seasons and the nature of oil it contains. In summer the rate of evaporation is more than in winter and lower fraction oil goes out and reacts more quickly than the higher fraction oil. By the actual

analysis of R. T. 3 and R. T. 4, it is found that the proportions of lower and higher fraction oils are almost 1 : 1 and 1 : 6 in average respectively.

Results of actual analysis of samples of R. T. 3 & R. T. 4 made according to I. S. specifications for carrying out the experiment are as given below :

	Characteristic	**RT 3**	**RT 4**
1.	Equiviscous temperatures (E. V. T)	44°C	55°C
2.	Softening point of residue (Ring & ball method)	44°C	46°C
3.	Moisture content	Nil	Nil
4.	Fraction (in gram per %)		
	(a) Below 200°C	Nil	Nil
	(b) From 200°C to 270°C	3.43	0.7
	(c) From 270°C to 300°C	4.14	5.56
5.	Phenols (by vol) %	Trace	Trace
6.	Naphthalene (by weight) %	0.1	Trace
7.	Matter insoluble in toluene % (C_1+C_2)	13	15

It has been found by a special type of cone penetrometer designed by the Author, Photo 4, that the hardening of R. T. 4 is slower than R. T. 3. The results of cone penetrometer readings on standard briquettes made out of R. T. 3 & R. T. 4 are given in Table 3.

TABLE 3

Readings taken after hours	Briquettes with grade of road tars	
	R. T. 3	R. T. 4
	Time taken in seconds for penetration	Time taken in seconds for penetration
0	7	26
24	15	41
48	25	56
72	87	103
96	150	150

From the results of the readings of the cone penetrometer plotted on Fig. 1 it is obvious that at the beginning R. T. 4 can stand higher temperature than R. T. 3 and in maturing upto certain length of time hardening goes almost side by side due to evaporation and atmospheric reaction on lower fraction oil. Afterwards, maturing of R.T. 3 and R. T. 4 coincides to a common point showing that R. T. 3, which was more soft than R. T. 4 in the beginning, has equalised with R. T. 4 after a certain period of time. From this it can be inferred that atmospheric reaction and evaporation of oil in R. T. 3 is more than in R. T. 4. As a result more life of road with R. T. 4 can be expected. On the other hand, R. T. 4 can resist more impact of the traffic wheel just after laying, saving thereby the chance of the surface being disturbed in the beginning as it happens in the case of Road Tar of lower viscosity.

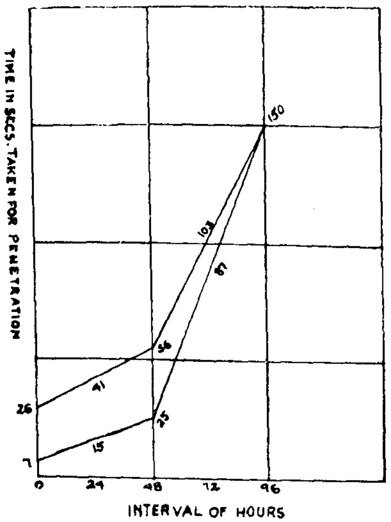


Fig. Sieve 1

To carry out this experiment, standard briquettes were made out of equal quantity of sand of same sieve value coated with equal quantity of R. T. 3 & R. T. 4 separately under uniform pressure given by a screw press designed by the Author, Photo 5. The penetrometer was designed with one C. M. cone at the head of the needle, Photo 4. The briquette was placed on a rigid support below the penetrometer cone and was kept in constant temperature. The cone point was then adjusted just to touch the upper surface of the briquette. After placing 1,000 grams load on the top, the penetrometer and the stop watch were simultaneously released and as soon as the cone point reached a depth of 1 C. M. of the briquette the stop watch was stopped and reading of time was taken. In this way readings on both the briquettes at equal interval of hours were taken and plotted on a graph. Fig. 1.

To find out the stripping effect on a particular type of stone, Riedel & Webber test was done both with R. T. 3 & R. T. 4. Result with R. T. 4 was found to be more satisfactory than the result with R. T. 3.

FIELD TRIALS

Besides laboratory results portions of waterbound surface of Upper Circular Road under Calcutta Corporation was treated with surface dressing using R. T. 4 (H. V. R. T.) and R. T. 3 (M. V. R. T) blinded with $\frac{3}{4}$ in. to $\frac{1}{2}$ in. Pakur Chippings. The road is a heavily trafficked one. On observation it was found that the portion with R. T. 3 worked up very quickly and bleeding occurred and they had to spread stone dust to avoid disturbance of the surface under traffic due to bleeding. The portion with R. T. 4 worked up slowly under traffic and kept the mosaic character of the surface for a long time even after working up of the binder through the voids of the chippings up to the top level of the same. After some time, in some of the places, the top surface of the chippings was partially covered with binder and the nice mosaic character was lost in those places. This might have been due to the uneven distribution of the binder at the time of laying. After passing about one year's traffic of all sorts, when the polishing out of the surface looked to be complete, the surface looked to be perfectly mosaic all over again and no sign of deterioration was found on the portion. The portion done with R. T. 3 looked to be fatty and there were signs of deterioration at the end of the 2nd year. After that the whole length was covered with 2nd coat and no further observations could be continued. The surface texture of the portion done with R. T. 4 looked to be ideal, because the chippings had very good bond one with the other as well as with the road surface and the top surface of most of the chippings was visible.

On our recommendation and request some miles of newly constructed waterbound surface under Development (Roads) Department, Govt. of West Bengal had been treated last year with initial coat of surface dressing using R. T. 4 as binder. Waterbound macadam was of Pakur Stone metal blinded with local earth. R. T. 4 heated to 240°F was spread by the help of pouring cans at the rate of 50 lb per 100 sq. ft. on an average and the uniformity of the film of binder was obtained as far as possible with the help of brushes. The painted surface was blinded with 5.5 cu. ft. of $\frac{3}{4}$ in. to $\frac{1}{2}$ in. Pakur Stone Chippings per 100 sq. ft. on an average. The Surface was rolled with 10-ton power roller. At the beginning traffic intensity on the road was not much. It gradually increased to some extent but still not up to the mark. Photo 6 will show from *A* to *D* how the gradual working up of the binder occurred with the gradual increase of the traffic intensity. There was no bleeding and no disturbances of chippings under traffic. The road surface temperature in mid-day was 110°F average. In case of surface dressing with R. T. 2 or R. T. 3 in many cases it was found that, on allowing traffic on the road after completion of the treatment, at mid-day when the temperature is high the surface was disturbed and in many cases bleeding occurred. But in case of R. T. 4 no disturbance of chippings under traffic was observed and all the chippings had good bond with the surface.

The question of comparison of cost of surface dressings done with R. T. 3 and R. T. 4 does not arise as the price of all the grades of Road Tars are the same, and the quantity required for both the cases is also almost the same.

CONCLUSION

To conclude, in consideration of facts and results of observations stated above, it is felt that under the climatic condition of India, more use should be made of Road Tar of higher viscosity (R. T. 4). If experiments or surface dressing with R. T. 4 are carried out on a well consolidated road, sufficiently strong to carry heavy traffic under controlled conditions, the results will prove the usefulness of this material both from the point of view of durability and riding quality.

ACKNOWLEDGMENT

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"PRESTRESSED CONCRETE PAVEMENTS"

By

K. K. BANERJEE*, B.E., C.E., M.S. (WIS)

CONTENTS

SYNOPSIS

In this note an attempt is made first to show why prestressed concrete should prove to be superior to any other form of concrete in road construction. Next, the various stress producing conditions are discussed. It is considered that no advantage whatsoever will be obtained by increasing the thickness of the slab beyond a certain limit. Lastly, descriptions of some of the methods of prestressing pavements which have been tried for investigation purposes with some suggestions are given.

1. INTRODUCTION

Very recently attention has been paid in Western countries to the possibilities of prestressing concrete pavements. It is believed that a prestressed concrete pavement will not only eliminate most of the defects of ordinary and reinforced concrete pavement but will also resist load and weathering condi-

*Bengal Engineering College, Calcutta.

tions better. For this, however, contrary to the common idea, the pavement will be made thinner and longer.

Reduction in thickness is possible because (i) prestressed concrete slab is stronger than ordinary or even reinforced concrete slab, (ii) reduction in thickness minimizes temperature and humidity stresses and stresses due to ground friction. Use of longer length is only possible when the slab is prestressed. Longer length of slab (about 400 to 500 ft) makes transverse joints few and far between. Hence improvement such as provision of foundation at the joint is within economic possibility. This procedure eliminates the inherent defects in a joint.

Of the various stress producing conditions, traffic load, friction of the subgrade and temperature and humidity gradients in a slab are the most important ones. In a prestressed slab, however, prestress will constitute another major stress. These stresses are to be taken into account in the design considerations and formulae for length of the slab and amount of prestress in the two right angle directions to be obtained.

After the last great war the French Airport authority wanted to build a runway at Orly for an unusually heavy traffic. The well-known French engineer, M. Freyssinet, who was given the job of designing it, had many problems to tackle including such heavy loads as 85 tons single wheel load. Being a pioneer and a great enthusiast of prestressing he had recourse to prestressing the concrete runway. With progressive research and tests he was convinced that prestressing was the best, if not the only, practical solution to this problem. After the runway was completed he carried out load tests on it which confirmed his earlier belief.

The research and tests at Orly opened certain lines of thought which were not quite realized before. In the light of these it would appear that if a pavement is ever made of concrete, prestressing is the best way to do so. There are certain defects of concrete pavements. Even highly reinforced concrete pavement is not free from all these defects. It is believed that prestressing will not only eliminate the defects but also allow the pavements to be built much thinner and longer than they are built at present.

So far the runway at Orly has proved to be a success. But the problems involved and the method of construction employed at Orly are only typical of runways. To apply the principles of prestressing as an economical and practical method of road construction considerable amount of research and investigations

will be necessary. Fortunately, however, attention has been already directed in most of the European countries and in the U.S.A to this problem and active research is under way.

2. WHY PRESTRESSING

It is a common knowledge that unless joints are provided at frequent intervals concrete pavements crack. These cracks may be due to various forces acting since the time of placing of concrete to the time of opening of the road for vehicular traffic and subsequent ageing. But generally the cause may be attributed to the following: traffic loads, shrinkage of concrete, temperature and humidity gradients within the slab, frictional resistance of the subgrade preventing free sliding, and uneven settlement or swelling of the subgrade.

The general tendency of the designer, in accommodating for higher loads, is to increase the thickness of the pavement slab. But an increase in thickness will give rise to greater difference of temperature and humidity, greater frictional resistance and will lead to bad use of ground resistance by increasing the thickness of the slab. Thus a thick slab is as inefficient as it is costly.

In a rectangular piece of slab, a load placed at the edges and corners gives rise to bending moments higher than those due to the same load placed at an interior point. The infiltration of surface water through the joints between neighbouring slabs causes a deterioration of the subgrade condition precisely at those edges and corners and further increases the moment. There has not been yet a perfect joint which could stop this menace and it is a common knowledge that these joints are expensive both to construct and to maintain. In addition, the impact of wheels damages the edges and the riding quality of the road suffers.

The only way to get rid of this defect is to do away with joints, or at least to have fewer joints. But this will give rise to high internal stresses. In the United States they have used continuous reinforcements to very long lengths of slab; but usually the reinforcements need to be very heavy and the steel becomes effective only after the concrete has cracked. To avoid cracks entirely, prestressing must be resorted to. Further, with prestressing, very long lengths of slab may be made crack-free and it will not then be at all uneconomical to provide foundations at the joints to avoid such dangerous moments.

Since dangerous moments at the edges and corners are taken care of and since for equal thickness prestressed slabs are stronger than ordinary ones, both in bending and shear, the thickness can be greatly reduced. This reduction in thickness in turn reduces temperature and humidity stresses, stresses due to ground friction and bending moment. The cumulative effect of all these factors allows curtailment in thickness of the slab to a considerable extent.

3. FACTORS CAUSING STRESS IN A SLAB

The various factors causing stress in pavement slab will now be discussed:

A. Stress due to Vehicular Traffic

A concrete pavement slab may be considered as a plate resting upon an elastic foundation, that is, as if the plate is supported throughout by closely spaced identical springs. There is a simple relationship between the deflection of the plate and the reactive presssure upon it by the foundation. But the solution of the problem involves exponential and hyperbolic functions and the process makes the general application very tedious. Some standard cases, however, can be solved and a given problem will be generally capable of being treated in the light of standard cases. H.M. Westargaard[1] has solved three cases under three critical load positions: (a) interior load, (b) edge load and (c) corner load. Since it is proposed to provide foundations at the edges and corners only the interior loading need be considered, in which case the maximum bending moment occurs under the load and acts radially in all directions.

B. Ground Friction

Any tendency of the slab to lengthen or shorten owing to climatic changes will be resisted by the subgrade. The subgrade applies a system of horizontal force by virtue of the frictional contact existing between slab and subgrade. The stress in the slab due to ground friction is maximum at the centre of the slab and is tensile or compressive according as the slab contracts or expands.

Though this force is acting at the bottom surface no account of its eccentricity should be taken, because the effect of eccentricity is just being counterbalanced by the dead weight bending moment of the slab, since the slab remains flat on the ground. The net effect is therefore a uniform stress throughout the entire section.

Now this stress is induced by the tendency of the slab to change its length either on account of temperature or moisture changes or both. But the magnitude of subgrade friction does not depend upon the magnitude of either temperature or moisture variation. For the latter to be the determining factor, the slab should have complete fixity to any change in length. This condition may arise in case of very long lengths of slab, say 2,000 ft.

In the case of ordinary reinforced concrete pavements with joints at usual spacings this frictional stress is of secondary importance. But since it is proposed to build quite long lengths of slab, the friction will be a major stress producing force and therefore attention should be paid to minimize its effect. A few inches of sand and some bituminous paper over the subgrade will appreciably reduce the frictional force.

C. Temperature Changes

The slab movement due to temperature change depends upon the difference in temperature at any particular instant and the temperature at which the slab was laid. If the slab is assumed to have been laid at some average temperature, a maximum plus or minus change may be taken to be about 60 degrees Farhenheit.

But the temperature changes, as they occur, are not of uniform intensity throughout the entire slab section. During the day the direct heat of the sun may quickly raise the temperature of the top surface much higher than that of the bottom and consequently there is a temperature lag between the surfaces. This differential temperature is transitory in character and goes through a reversed daily cycle. The causes which give rise to such a phenomenon are obviously poor conductivity of concrete and sudden heating or cooling of the top surface. Observations on different thicknesses of slab during different seasons of the year show a maximum differential temperature as:

During the day	$+ 3°$ F per inch thickness of slab
During nightfall	$- 1°$ F ,, ,, ,, ,, ,,

The temperature change therefore causes two things, namely,

(i) an overall increase or decrease in length, giving rise to frictional stress, which has been already dealt with, and

(ii) 'warping stress' due to temperature difference between the surfaces.

Warping Stress—The temperature gradient between top and bottom surfaces of a slab may be assumed to be linear. In a free structure a linear temperature gradient would induce stress-free bending. But in this case the dead weight of the slab will resist bending, making the slab lie all the time on the subgrade. Assuming the top surface to be at a higher temperature than the lower, the slab will tend to bend with convexity upwards. The dead weight prevents such upheaving and therefore compressive stress is induced at the top fibre. The reverse will happen when the upper surface is cooler than subgrade temperature.

In a very short slab there may be some relief of this stress due to the fact that the slab may, in that case, take some appreciable curvature throughout its entire length. But in a long slab the warping stress is not influenced by its length.

D. Moisture Changes

Concrete undergoes volumetric change with change of moisture content. It swells if kept under water and contracts when it is drying. As a matter of fact any porous material behaves just the same as concrete. This behaviour may be explained as follows:

The water held in the fine pores of concrete is separated from air which fills the non-wetted part of these pores by menisci which support high surface tension. The water thus enclosed is under tension which seems to follow the laws of Thermodynamics. Accordingly, the tension will be some function of humidity. Since this tension is resisted by the net section of concrete, the three dimensional pressure on concrete will be proportional to the ratio of pores to the net volume of concrete. This pressure from all directions causes the concrete to shrink and the shrinkage at any moment is a function of water contained in concrete and the humidity of atmosphere. It is obvious from the above argument that a poor concrete will shrink more.

Concrete undergoes variation in moisture from the moment it is mixed with water. During the hardening period it loses a large portion of excess water initially required for working. This reduction in moisture content tends to shrink the concrete but the subgrade friction offers resistance to such shortening and thereby induces tensile stress. Though the induced stress may be relatively small as compared with subsequent stress induced during service, it is harmful and may cause cracks since it tends to occur at a time when the concrete is very weak. To avoid such cracks concrete should be effectively cured till it has attained sufficient strength.

After the slab has undergone a suitable hardening period, it acts as a rigid mass and although similar hygroscopic changes occur, it is in a better position to resist them. Stresses may then be analyzed for a concrete of known resisting properties.

Unlike temperature change which alters the length of a free structure without causing any stress in it, alteration in size due to hygroscopic change, as has been shown, is always accompanied by some internal stress. Over and above this internal stress there will be some warping stress induced by differential moisture content across the thickness of the slab, just the same way as it is with differential temperature. The frictional force of the subgrade due to variation in length need not be considered separately in this case.

If, therefore, definite values of moisture content in the different parts of a pavement slab could be reliably or accurately determined, as in the case of temperature, it would no doubt be possible to determine the stresses induced thereby. But unfortunately the behaviour of such a moisture gradient is to some extent problematic. However, it may be possible to guess some reasonable values of the moisture gradient and assume it to vary linearly across the thickness. The total shrinkage strain of concrete after a lapse of considerable time of drying amounts to 300 to 600 micro in. per in., depending upon the quality of concrete. When water is poured over dry and set concrete it quickly absorbs water and soon recovers a considerable part of total shrinkage. Therefore, in the event of prolonged drying and then being suddenly wetted as by rain water, a pavement slab may in the worst case be subjected to a maximum average differential strain of the order of 200 micro in. per in. between the top and bottom surfaces. Assuming this to be linear and also assuming that the top surface is wet while the bottom is dry, the effect is similar to higher temperature of the top surface. Since, again, wetting will obviously bring down the temperature of the top surface, these two phenomena will be generally opposing each other.

A differential strain of the order of 200 micro in. is equivalent to a temperature lag of 36° F, or about 6° F per in. for a 6 in. thick slab. Hence it is seen that the effect of humidity is more severe than temperature effect on a slab. This fact was not quite realised before. However, considerable research and tests will be necessary before the exact magnitude of this is known.

E. Other Stress Producing Conditions

There are also other factors which may produce stresses, under adverse conditions, of very high magnitude, as for example ; (a) non-uniform subgrade condition and (b) volumetric changes in the subgrade. Since effects of these indeterminate conditions cannot be reliably evaluated in terms of computed stress, they must necessarily be eliminated from rational considerations and thrown upon the 'factor of safety' to take care of.

Critical Stress Combination

It is a matter of common observance that a slab tends to lose moisture while it is accumulating progressive rise of temperature. Also the variation of temperature in a day during a rainy weather is much less than it is in a dry day. Therefore, the effects of moisture and temperature naturally tend to oppose each other. Hence in design considerations it will be sufficient to provide for the maximum of either of them.

The traffic load will be cumulative to warming or wetting of the top surface. Regarding frictional stress it will produce compression while the slab is gaining temperature or getting wetter and therefore will increase the value of total compressive stress and reduce tensile stress. In the case of long slabs the frictional force may be considerable and it may cause maximum tension when the slab is cooling.

4. DESIGN METHODS

Several investigators had attempted to find a design method for this kind of pavement of whom F. Leonhardt [2] of Germany and P. B. Morice [3] of England are noteworthy. Leonhardt derives the magnitude of prestress in a concrete pavement from the consideration that the latter must be able to overcome the stresses arising out of frictional resistance and average temperature gradient. For live load stresses and for occasional extreme temperature gradient he relies on the tensile strength of concrete, because these stresses are temporary in nature and if cracks are formed they will close again when extreme condition ceases. On this consideration he suggests a slab to be made 16 cm. (6¼ in.) thick with joints spaced at about 150 metres (520 ft) apart and the slab stressed to about 20 kg per square cm. (284 lb per sq. in.). He also suggests a method of prestressing which is described later.

P. B. Morice in his design procedure assumes constant values of subgrade friction of 1 lb per sq. in. and stress due to temperature gradient as 100 lb per sq. in. From the consideration of maximum limiting compressive stress he obtains the maximum

permissible length of the slab. Fig. 1 shows the relation he obtains between the slab length and the percentage of high tensile wires for different thicknesses of slab.

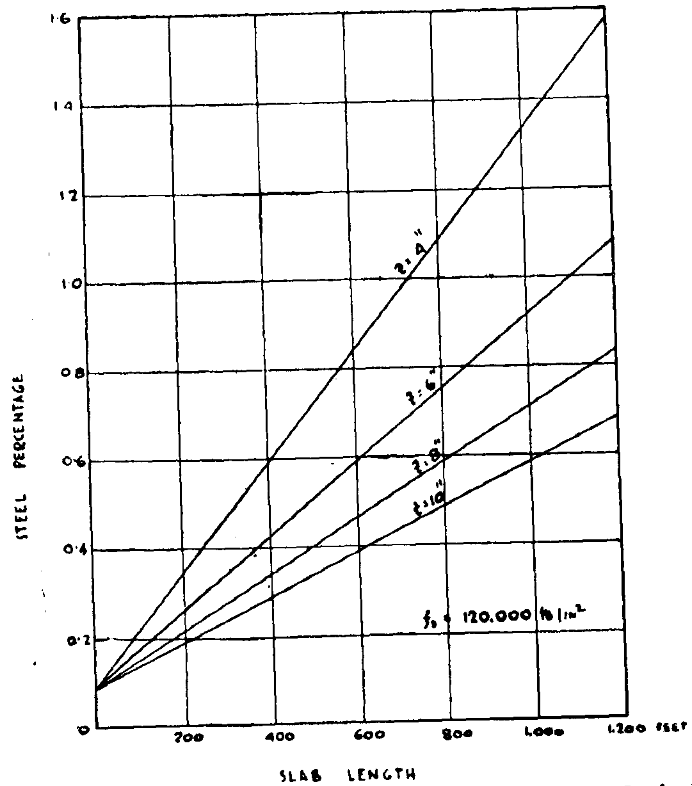


Fig. 1. **Percentage of steel against length of slab (Morice)**

Both Leonhardt and Morice based their design method on some average value of friction and temperature gradient stresses irrespective of slab thickness. This, it is considered, has failed to take advantage of the fact that a reduction in thickness of the slab leads to an overall saving. The complete omission of traffic load stresses in their design is certainly subject to criticism. The effect of humidity, which has not been taken into account by them, is generally greater than temperature effect and therefore the question of humidity deserves consideration and careful study.

On the other hand a frictional force of 1 lb per sq. in. of the ground area, as has been assumed by Morice, would be considered to be very high. This value, which has been obtained by Sparkes[4] as an average value on a series of experiments carried out by him, is probably taken on slabs cast direct on ground. But with a few inches of sand and bitumen paper, a coefficient of ground friction of about 0.5 as has been estimated at Crawley New Town[5], seems to be reasonable.

In attempting to make a rational approach to the problem of design the following determinate stress producing conditions may be incorporated in the method of design :

(a) Stress due to traffic load;

(b) Frictional stress;

(c) Warping stress; and

(d) Prestress.

All the above factors are dependent upon the thickness and length of pavement slab. Since theoretically almost any thickness of slab can be effectively prestressed to suit any loading or variation of temperature and moisture, it will be advisable to assume a thickness of slab from practical and economic considerations. Thickness being assumed, the warping stress and the stress due to any particular traffic load can be easily found out. Hence prestress and frictional stress (frictional stress depends upon the length of the slab) remain to be found out. For this, two equations may be framed, one based upon the maximum compressive stress that may be allowed in the slab and the other on the permissible tensile stress.

It may be observed that while resistance to bending increases with the increase of slab thickness, the stresses due to various other factors also increase. Hence there is a limiting thickness of slab beyond which no advantage whatsoever is gained. From the foregoing theoretical considerations the following generalisations are arrived at :

1. Any thickness of slab can be successfully prestressed to suit any loading and variation of temperature and moisture.

2. For thinner slabs live load stresses are more but induced stresses are less. The reverse is the case for thicker slabs.

3. The maximum length of a slab for any particular thickness of it increases with the increase of the strength of concrete.

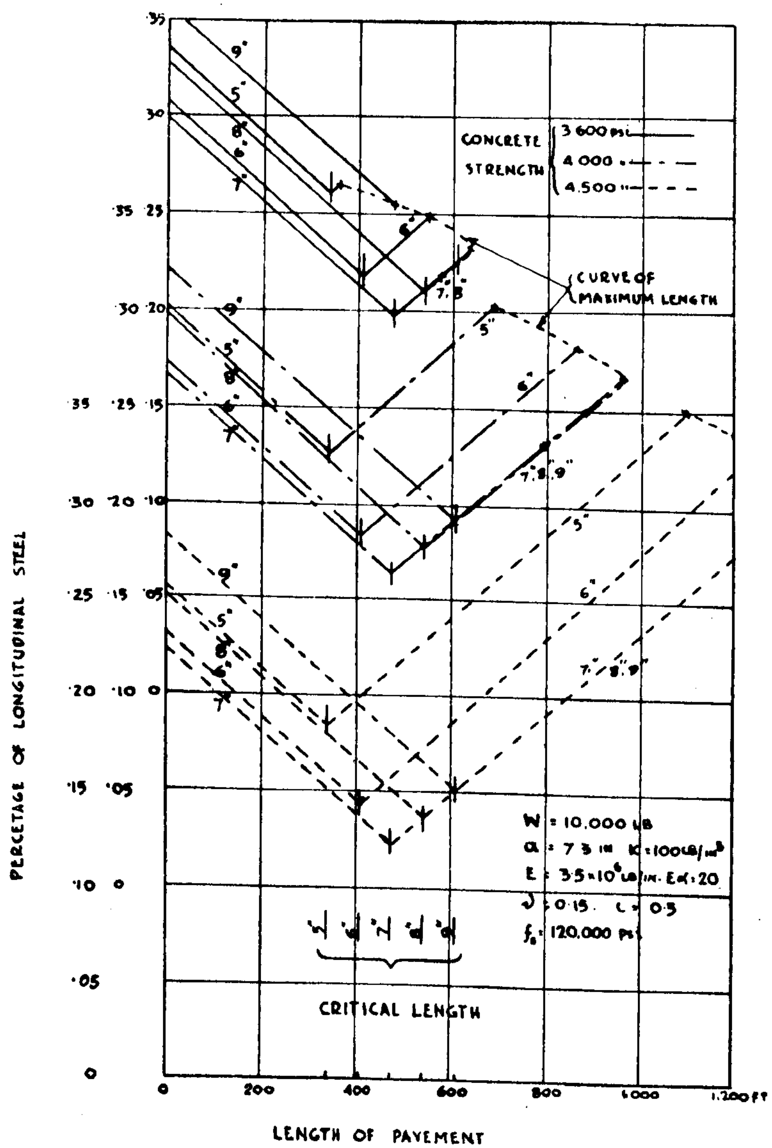


Fig. 2. **Percentage of steel against length of slab**

4. For the same strength of concrete, thicker slabs permit longer lengths to be employed.

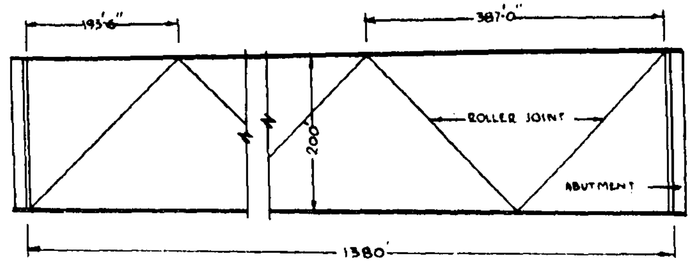
Fig. 2 shows the percentage of high tensile steel for different lengths and thicknesses of slab. The maximum limit to the length of slab for various strengths of concrete is also indicated. It may be noticed in Fig. 2 that there is a critical thickness of slab, about 7 in., above which no advantage whatsoever is gained. Inspite of the very heavy load, Freyssinet employed a thickness of only 16 cm. ($6\frac{1}{4}$ in.) at Orly, which then appeared to be too thin to almost everybody. It may now appear how correct Freyssinet was in choosing the thickness.

5. THICKNESS OF ROAD SLAB

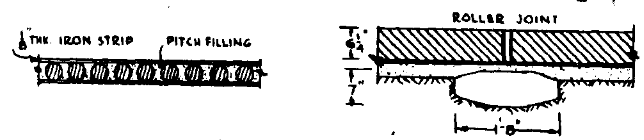
Though it is economical to use a thinner slab, a 4 in. slab is to be used in a restricted manner. It does not permit use of long slabs except with exceptionally good quality concrete. A 5 in. slab seems to be the answer for road construction. But if Freyssinet cables are used, to permit use of their anchorages, the edges where they are provided should be made at least 6 in. thick. Cables should run through the centre height, though it does not make any difference, so far as prestress is concerned, where the cables are placed and the effect will be always an uniform prestress throughout the entire section. If cables cross one another, a 5 in. slab will be on the tight side, since it leaves a cover of only $1\frac{1}{4}$ in. to the cables.

6. METHODS OF CONSTRUCTION

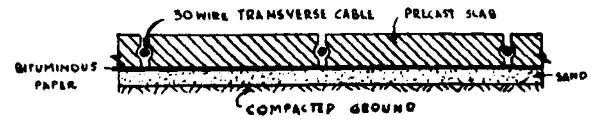
A. Orly Airport Runway. The method of construction used at Orly is unique in many respects. The runway is 1,380 ft long and 200 ft wide and is constructed of metre square precast concrete slabs $6\frac{1}{4}$ in. thick laid on a mixture of sand and clayey soil. The runway is divided diagonally by roller joints in the shape of isosceles triangles. There are two prestressed concrete abutments firmly anchored into the ground at each end of the runway. Transverse cables of 30–5 mm wires were laid in-between the rows of slabs and tensioned to 69 tons per sq. in. The tranverse prestressing acting on the roller joints induces a longitudinal prestress and is held between the abutments. The longitudinal prestress thus created serves to eliminate the effects of temperature and shrinkage cracks. Fig. 3 shows the details of some of the features of the design and the general arrangement of the runway.



Plan of runway at Orly Airport near Paris



Steel rollers fill the joints   Concrete supports beneath joints



Recessed edges allow cables to pass between slabs

Fig. 3. Runway at Orly

B. Method Proposed by Leonhardt. He proposes to have straight cables laid in protective sheaths and concreted in position. They are to be stressed later by jacks mounted on some rubber tyred vehicle of such capacity as to force apart the edges of a specially made transverse joint, Fig. 4. After stressing operation the two sections of slab are held in position by spacer blocks and the gap concreted.

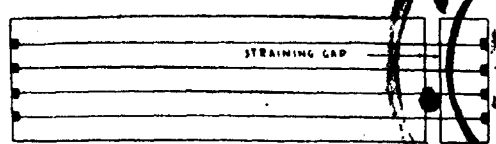


Fig. 4. Leonhardt's method of prestressing pavements

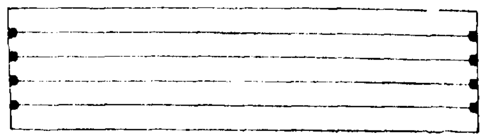


Fig. 5. **Straight cables at Wexham Springs**

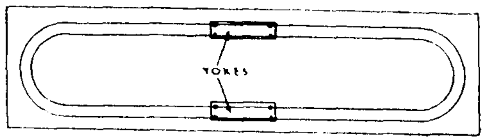


Fig. 6. **Perimeter cables at Wexham Springs**

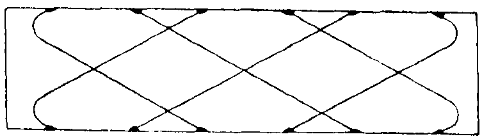


Fig. 7. **Diagonal prestressing at Crawley New Town**

C. Method used at Wexham Springs. The Cement and Concrete Association, U. K., laid two experimental sections at their research centre at Wexham Springs and used different methods in each case. In one of the sections, Fig. 5, the most obvious method of running cables from end to end had been adopted. Ten number 12-0.2 in. Freyssinet cables were used for a width of about 15 ft. In the other case, Fig. 6, two cables are run along the perimeter and stressed against two yokes near about the middle.

D. Road at Crawley New Town, Fig. 7. Cables are run diagonally across the slab in protective metal sheaths.

7. RELATIVE MERITS AND DEMERITS OF DIFFERENT SYSTEMS

The system used at Orly runway seems to be the most practical and economical one so far as runway is concerned. Being of some definite length a runway could be provided with end abutments to obtain longitudinal prestress. Certainly this method cannot be applied in the case of road slab. It is a

good idea to have precast elements which save time at site. Though the elements in the case of Orly were precast at the site, in case of standard design, these can be better cast in a factory, transported to site, laid in position and stressing operation completed within a very short time. It is certainly worth considering the use of precast elements in the construction of roads.

The Leonhardt method can only be used with straight cables and therefore no transverse prestressing can be obtained. The system does not impress very much also because of the fact that it calls for a heavy stressing equipment. Further, two casting operations, one for the pavement and the other for the gap, will delay the work.

The end-to-end cables laid at Wexham Springs suffer from the same disadvantages as the Leonhardt system except for a lighter jack. But on the other hand, to be able to use the conventional type of jack at the end of the slab, a gap must be left between the consecutive lengths of slab and concreted later.

The perimeter cable has a very limited application. This system may be used in the case of short lengths but cannot be trusted in the case of longer lengths, because, while the entire section is to be prestressed, and particularly where the centre section is the section of maximum stress, perimeter cables alone may not induce the desired prestress on account of ground friction. Also, taking the cables round the curve will entail considerable loss of prestress[6].

The method of diagonal prestressing as used at Crawley New Town is one which is meant to provide prestress in both the directions. In all other cases, prestressing in the transverse direction has not been worked out, which in effect is equivalent to using plain concrete section in that direction.

Slabs in which both the sides are approachable can be stressed from either side. When one side is not accessible, say where there is a longitudinal joint, the end of the cables on that side can be exposed and fanned out for bond while stressing is carried out from the other end, Fig. 8. As a matter of fact, to save anchorage cost, even if the side is accessible, this procedure may be employed. The advantages of diagonal prestressing may be summarised as follows: (i) both transverse and longitudinal prestressing are created simultaneously, (ii) no second casting is necessary, (iii) less anchorage cost, (iv) length of cables being

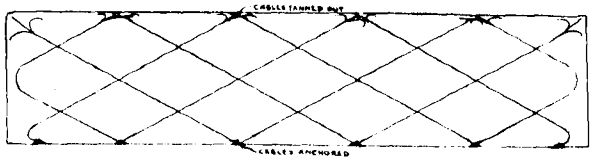


Fig. 8. **Diagonal prestressing ; Anchorage at one end only**

short, they are easy to manufacture, transport and use at the site, (v) capable of negotiating curves better, and (vi) almost no friction loss.

8. SHRINKAGE CRACKS AND CONSTRUCTION JOINTS

Though the length of slab is proposed to be very long, construction and concreting should be carried out in section to avoid shrinkage cracks. Experience has shown that construction joints should be left at intervals of about 50 ft and in the case of wide slabs, a longitudinal joint at the centre. Concrete should be effectively cured till it has attained sufficient strength.

9. EXPANSION JOINTS AND EDGE FOUNDATION

Observation at Crawley New Town has shown that expansion of 400 ft long prestressed slab is not sufficient to warrant the construction of complicated joints. It seems reasonable to provide normal form of joint with a total width of 1 to $1\frac{1}{4}$ in. A precast concrete element as used at Orly runway, Fig. 3, or a dwarf brick wall may be used with convenience to form the foundation at the expansion joint and at the edges. Some form of precast concrete kerb may be alternatively provided at the longitudinal edges to function both as a kerb and foundation to the edges.

10. COST AS COMPARED WITH NORMAL CONSTRUCTION

The future of this type of construction of roads depends very much upon how competetive it is with respect to normal forms of construction. At Crawley the subgrade conditions fixed normal construction as 12 in. of concrete comprising a 4 in. lean concrete sub-base and 8 in. main slab of good quality concrete. In the prestressed case a 6 in. slab was adopted without the sub-base. Owing to the experimental nature of the work no comparison could really be made but subsequent investigation revealed that the prestressed concrete road could be constructed at a cost comparable with normal construction. This is more so where the subsoil is weak and requires some heavy form of normal construction.

11. CONCLUSIONS

It is clear from what has been said and stressed earlier in this note that when concrete is prestressed it makes the best form of concrete pavement because it permits (a) reduction in thickness of the slab even in a very poor case of subgrade condition, (b) use of long length of slab as 400 ft thus eliminating frequent transverse joints and (c) use of concrete without any crack at all. Evidently a pavement like this is expected to last longer without much, if at all any, maintenance cost.

Emphasis has been also laid on the fact that there is always a limiting thickness of slab above which no advantage whatsoever is obtained. A similar limiting condition is also present in the case of ordinary or reinforced concrete pavement but on account of insufficient knowledge it had been the practice in the past to increase the thickness out of bounds to accommodate higher loads.

While constructing prestressed concrete pavements it will be worthwhile paying attention to the following points : (1) There should be no tendency to increase the thickness of slab beyond the limit that will be necessary from practical and economical considerations. (2) A convenient workable length of the slab should be adhered to (say about 400 ft). (3) A practical and economical method of construction has to be evolved to compete with other forms of construction—which, again, can only be attained with increasing use of this type of construction. (4) As climatic conditions very much influence both design and method of construction, local experiments should be carried out to find out the best values of thickness, amount of prestress, length and other factors.

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RESEARCH SECTION

"ABSORPTIOMETRIC ANALYSIS OF ASPHALTIC BITUMENS"

By

C. G. SWAMINATHAN*

&

S. ARUMUGA SWAMI*

CONTENTS

SYNOPSIS

Based on the optical absorption characteristics, an analysis of asphaltic bitumens normally used for road making purposes is attempted. A Hilger-Spekker photo-electric absorptiometer was used in the investigation and the extinction co-efficient based on the slope of the curve connecting optical density and concentration was adopted as the main distinguishing factor. On the assumption that this extinction co-efficient is constant for a given wavelength, it is found that the absorption characteristics bear no direct relationship to the normally employed physical properties of the bitumen but can only be related to the nature and the chemical composition of the constituents. Because of this, the changes in the structure of the bitumen as a result of weathering, overheating etc. can be detected, by this means. Also, since the extinction constant is very much influenced by the source and the method of manufacture, the absorptiometer can be used as a useful tool for purposes of identification of bitumens to a limited extent.

1. INTRODUCTION

It is wellknown that the structure and the chemical composition of asphaltic bitumens are quite complicated and varied.

* Central Road Research Institute, New Delhi.

Their properties, both physical and chemical, are very much influenced not only by the nature of the source of the crude but also by the method of manufacture and by the refining processes to which it is subjected. Simple tests for the choice of the material to meet the various conditions of usage do exist. But the test values give only a measure of consistency and allied properties, facilitating an approximate differentiation of one material from the other and they are not quite satisfactory from the point of view of definite identification. Microscopic and X-ray tests have also been conducted to study the characteristics of bitumen, but only to a limited extent.

In addition to all these tests, the colorimetric method of analysis has also been attempted. The first, probably, to employ this method for identification of bitumen in paving mixtures was Wilson[1] who employed a Lovibond visual colorimeter. This suffers from the drawback that the colour of the solution under test is determined by matching colours by visual inspection. Broome[1] later on used the photo-electric colorimeter quite successfully for the same purpose by which various types of binders could be identified, and the changes that take place in the bitumen as a result of weathering or over heating could also be detected. He expressed the test values in terms of the potential difference built up in the photo-electric circuit by the light emerging from the bitumen solution. But it is felt that the method of expressing the data in terms of chromatic values is rather arbitrary. Also the effect of different solvents and the influence of the different concentrations of bitumen in the solvents have not been gone into by Broome. Hence in this Paper it is attempted to study these aspects in greater detail and also to express the test values in terms of a definite light absorption characteristic of the bitumen, namely, the extinction constant of the material.

2. THEORY OF ABSORPTION

The basic laws governing the absorption of light were formulated by Lambert (1760) and Beer (1852) as follows.

Lambert's law states that "each layer of equal thickness absorbs an equal fraction of the light which traverses it."

Beer's law states that "the absorption depends only on the number of absorbing molecules through which the light passes."

In the light of these two laws, it can be stated that "the optical density of a coloured solution measured with monochromatic light is proportional to the number of molecules absorbing the light, in other words, proportional to the concentration of the solution[4].

Speaking mathematically, when a beam of light of intensity I passes through an infinitely small thickness 'dl' of a solution of an absorbing substance of molecular concentration 'c' expressed in grams per litre, the ratio of the absorbed and incident light (if the amount of light absorbed be 'dI') can be expressed as $\frac{dI}{I} = -kcdl$ ....(1) where k is a constant for a given wavelength[2]. Here it should be noted that the amount of light absorbed depends on the wavelength of the light and the constant k will be different for different wavelengths

Now, if the light is absorbed by a layer of thickness 'l' and if Io and I are the intensities of the incident and emergent lights respectively, then integrating expression (1) and taking suitable constants, it is obtained that

$$\log \frac{Io}{I} = D = E.C.\,l \quad \text{............(2)}$$

Where D is known as the optical density.

E is the extinction coefficient or extinction constant.

C is the concentration of the substance in gms/100 cc.

l is the thickness of the layer in centimetres.

3. THE APPARATUS

Photo-electric colorimeters are instruments in which the amount of light absorbed by a coloured solution is measured by allowing the transmitted light to fall on to a photo-electric cell. In general, they fall into two classes according to whether they employ one or two photo cells. Theoretically, the two-cell type is more satisfactory since the two cells, which are carefully matched, work in opposition and any extra influencing factor would affect each cell equally and cancel out. Further, they would be more sensitive. It is for these reasons that the Hilger Spekker absorptiometer, Fig. 1, together with a spot galvanometer was chosen for this investigation.

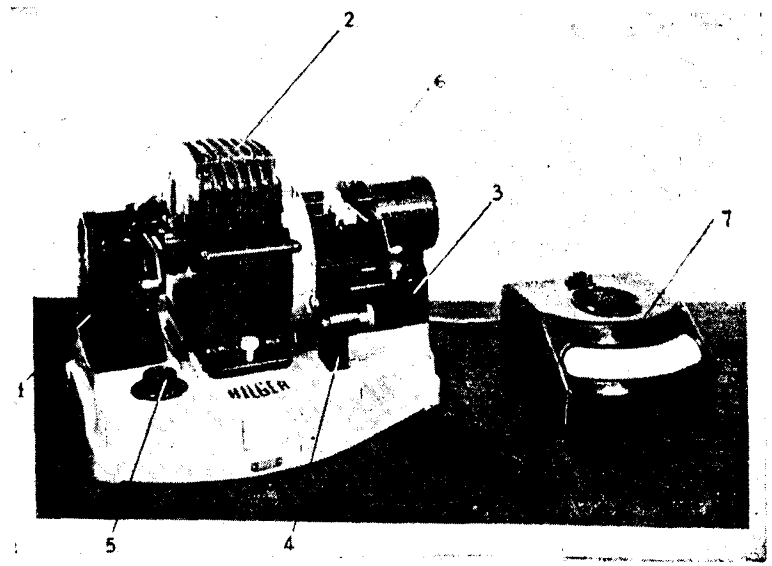


Fig. 1.

1. Left Photo Cell.
2. Lamp House.
3. Right Photo Cell.
4. Slow Motion Box.
5. Sensitivity Control.
6. Glass Cells.
7. Spot Galvanometer.

It is of interest to point out here that the Spekker absorptiometer has been used for "photo-electric methods in clinical biochemistry" by G. E. Delory and for "Metallurgical Analysis" by F. W. Heywood & A. A. R. Wood.

The absorptiometer, Fig. 1, is mainly composed of two photo-electric cells at ends mounted in black mouldings and a light source (100 W. Tungsten Filament lamp) in the centre. Between the bulb and the left photo cell there is an iris diaphragm, by means of which the amount of light falling on the left photo cell can be adjusted. Between the right photo cell and the lamp bulb there is a drum with a calibrated aperture in its centre to allow light to pass through and also a cell mount made of polythene in which are placed the glass cells containing the solutions to be tested.

The highly sensitive spot galvanometer is connected to the photo cell circuit by a jackplug at the back of the absorptiometer and is controlled by a potentiometer and a "sensitivity control" fixed in the front of the absorptiometer. The complete circuit is shown in Fig. 2.

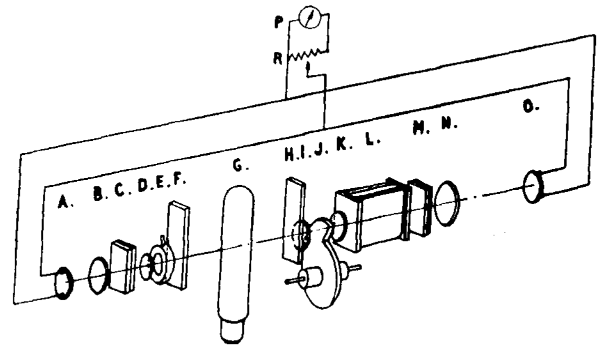


Fig. 2. **Simplified diagram of spekker absorptiometer optical system and photo cell circuit**

A and O	Photo cells.	I	Window.
B	Lens.	J	Measuring Disc.
C	Spectrum Filter.	K	Lens.
D	Window.	L	Cell for Liquids.
E	Iris Diaphragm.	M	Spectrum Filter.
F	Heat Absorbing Filter.	N	Lens.
G	Lamp.	P	Spot Galvanometer.
H	Heat Absorbing Filter.	R	Sensitivity Control.

The galvanometer spot will rest at zero, if equal current is produced in the two photo-electric cells which work in opposite directions.

The calibrated drum directly records the optical density of the solution. To transmit light from specified regions of the spectrum, eight pairs of gelatine filters, ranging in colour from violet to red, Table 1, are also used and the test is carried out under any one pair of filters at a time. On the right side of the absorptiometer, the filter is introduced between the photo-cell and the solution and on the left side between the iris diaphragm and the photo cell.

In the absorptiometer, the amount of light absorbed by the solution is determined by comparing the sizes of aperture required to give the same intensity of emergent light, when the incident light passes through the same thickness of the solution and the solvent respectively.

In use, the instrument is so set that when the aperture is fully open and the solution is in the path of light, a zero galvanometer reading is obtained. The solvent is now substituted for the solution and the galvanometer reading is restored to zero by decreasing the size of the aperture, which is varied by means of the drum, calibrated in the following manner.

If the area when fully opened be A and the area at another point be B, then the drum reading at that point gives the value of $\log A/B$. Since the area A can be regarded as a measure of the intensity of the incident light and B that of the transmitted light, the drum reading is a measure of the optical density of the solution.

4. BITUMENS TAKEN FOR THE STUDY

The various types of asphaltic bitumens could be classified under three groups, namely (a) Straight run, (b) Blown bitumen, and (c) Cutback. The straight run bitumens can further be sub-divided into various grades based on the standard penetration values. The blown bitumens can be classified on the basis of penetration and softening points, while the cutbacks could be grouped based on the curing periods. Since blown bitumens are not used for road making purposes, only one type of blown bitumen has been included in this study. Also, only one cutback grade (which is more predominantly used in India) has been taken up in this investigation. The data for these two would serve for purposes of comparison between the relative behaviour of the three types of bitumens. As far as the straight run bitumen is concerned, a more or less complete range (based on standard penetrations) of asphaltic bitumens has been taken up, in order to study their comparative absorptiometric characteristics. Also, since it is well known that the nature of an asphaltic bitumen depends to a great extent on the nature of the original crude and the method of manufacture, the bitumens that were studied in this investigation were obtained from three different firms. In all 12 different types were tested in this investigation and they are given in Table 2. The standard physical properties of these bitumens have also been given in Table 3.

5. TEST PROCEDURE

The general method of determining the absorption value of the bitumen consists of making a solution of bitumen of known concentration in a suitable solvent. Both the solvent and solution are tested on the absorptiometer. Benzene was employed as the solvent in this investigation since it has been found by

Broome[1] that "benzene solutions are far more stable to light etc. than solutions in any other solvent". However, in order to find out as to whether the nature of the solvent would influence the absorptiometric characteristics or not, one test was carried out using Trilene as a solvent. To minimise the effect of light on the solutions, the tests were conducted as quickly as possible and the solutions were kept in the dark as long as feasible.

Since the concentration of the prepared solutions is usually very low, the sample of bitumen is very accurately weighed and dissolved slowly in cold benzene and the quantity is made up to 100 cc. exactly. The concentration of the solution is expressed in grams of bitumen in 100 cc. of the solvent. Two glass cells of equal thickness are then taken and one is filled with the carefully prepared solution while the other is filled with the pure solvent. The thickness of the cells used in this investigation is ½ cm. In order to check the effect, if any, of the thickness of the glass cell, a single run of tests was conducted on a 1 cm. thick cell also, Table 5.

The two cells are then kept in the cell mount in the right hand side of the absorptiometer. The spot galvanometer is connected to the photo cell circuit and the switch is put on. The spot is then adjusted to rest at zero. Heat absorbing filters are then introduced in the light path on either side of the lamp source Fig. 2. The pair of coloured filters are inserted in the two mounts one on either side. The iris diaphragm is closed and the drum is adjusted to zero to give the maximum opening of the aperture. The cell mount is so adjusted that the glass cell containing the bitumen solution is in the path of light. The switch of the lamp is then put on and the gravity shutter of the lamp house is then pulled down to allow light to pass through. This light passes through the bitumen solution which absorbs some light and then falls on the right hand photo cell only, thereby generating a current to establish a potential difference causing a deflection towards the right in the galvanometer. To make the instrument sensitive for accurate readings, the sensitivity control of the instrument is adjusted in such a way that the deflection of the galvanometer is exaggerated by making the spot move towards the extreme right. At this stage the iris diaphragm on the left hand photo cell is slowly opened, causing the generation of an opposing current to nullify the original potential difference in the potentiometer. This makes the spot of the galvanometer to move towards the left. The iris diaphragm is adjusted till the spot returns to zero. The lamp house shutter is now closed and the cell mount is adjusted so that the cell containing the clear solvent is brought in the path of light. Since the clear solvent will absorb less light than the bitumen solution, more light falls on the right hand photo cell, thereby causing the spot to be disturbed from zero and again move towards the right end of the galvanometer. Now the drum is rotated clockwise to narrow down the opening of the aperture so as to cut out the intensity of light passing through. The drum is rotated until the spot of the galvanometer is restored to its zero position. The drum reading at this stage is recorded and (as described in section 2) the optical density of the solution is directly obtained.

For any one concentration of the bitumen solution the test is repeated for eight different pairs of colour filters varying from violet to red as well as white light, Table 4. This constitutes a series or one single run. In this manner, a series of tests were conducted for different concentrations of the bitumen solution for each of the bitumens taken and for each solvent used.

The range of concentrations chosen in this study is from .01 to 0.13 for 100 cc. This range was chosen because with low concentrations errors were likely to creep in owing to the exactness in weighing while higher concentrations make the solutions so dense as to affect accuracy.

The extinction constant is then calculated using the equation (2), page 111.

$$\text{Extinction constant } E = \frac{D}{Cl}$$

Where D = Optical density recorded from the drum expressed as a number.

C = Concentration of the bitumen solution expressed in gms. per 100 cc.

l = Thickness of the glass cell in cm.

E will have a dimension of cc/gm.cm. and in this note it is the extinction constant E which is adopted as an indicating factor for the light absorptive characteristics of the bitumen.

The following Tables and Figs. 3 to 5 were drawn up for facilitating the study.

Table 1: gives the details of the colour filters used.

Table 2: gives the various grades and nomenclature of the bitumen used.

Table 3: gives the 3 basic physical properties of the bitumens usually employed in the road making industry for choice of bitumen.

Table 4: gives a typical data sheet giving the optical density D and the extinction constant E for the different filters for varying concentrations for the different grades of bitumen. The last but one column in the Table gives the co-efficient of variation and the last column gives the E values as measured from the slope of the D & C curves at the origin. To avoid repetition and since the test results are similar only one typical data sheet is included.

Table 5: gives the E values for two different thicknesses of the cell, but using the same bitumen and the same solvent.

Table 6: gives the E values for two solvents, but using the same bitumen and the same thickness of cell.

Table 7: gives the E values as measured from the slope of the D & C curves for the various grades used in this study.

Table 8: gives the grouping of the various grades based on the E values.

Figs. 3 to 5: are the graphs connecting D & C for the various filters for typical grades of bitumen namely the penetration grade (Fig. 3) the Blown grade (Fig. 4) and the cutback (Fig. 5). Since the curves for the penetration grades are similar in shape (only different values of E as shown in Table 7) only one typical case is included in the note.

6. DISCUSSION OF RESULTS

A. Fundamental Considerations

While studying the absorption characteristics of the bitumen solution it should be borne in mind that the Beer-Lambert law is applicable to monochromatic light only and that each of the colour filter used has different transmission properties with its own range of wavelengths.

Further, the following four phenomena will also influence the absorptiometer readings, namely:

(a) Flourescent variations in the solution

(b) Scattering of light within the solution

(c) Reflection and refraction of light within the cell.

(d) Changes in the nature of the absorbing molecules.

In this note, the above four factors have not been taken into consideration separately and it is assumed that their influences on the test values could be considered negligible and neglected for all practical purposes, especially since the main object of this investigation is for identification of bitumens rather than in the form of a fundamental study.

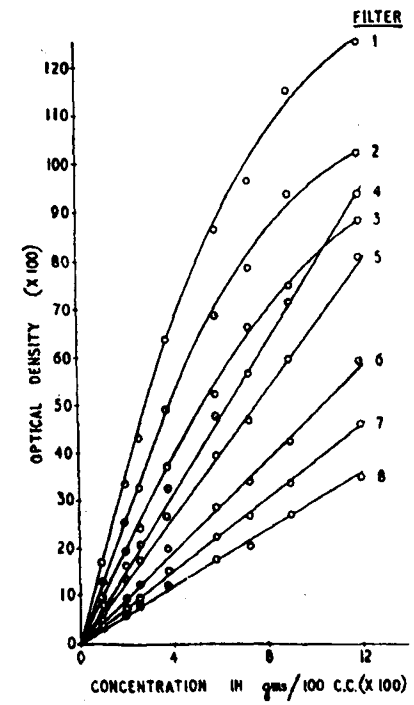
From the various Tables and Figs. the following observations could be made:

(i) For any bitumen, with a given solvent, and given thickness of the cell and for a given concentration the values of both D and E are different for different filters. Both of them go on decreasing with an increase in the wavelengths of the light passing through. The slopes of the curves connecting D & C at the origin, giving the values of E, also show the decrease of E with an increasing wavelength. This is only to be expected since according to the law of absorption both optical density and the extinction constant should change with the wavelength of the light used. For, since $D = \log Io/I$, D should vary with the ratio of the intensity of the emergent and incident lights which in turn would be influenced by the absorption characteristics of the solution, being different for different filters.

(ii) For any given bitumen, with a given solvent and given thickness of the cell and for any given filter, both the values of D and E are different for different concentrations. Within the range of concentrations chosen in this study, it can be seen from Table 4 that the optical density increases with the concentration. This is only to be expected since in the equation $D = ECl$, if l is fixed and E is assumed as a constant, D should vary directly with C.

From Table 4 it is seen that there are variations in the value of E itself. These variations are more marked for filters 1-3 while for filters 4-8 the variations are not much. From the co-efficient of variations given (in the last but one column in the Table) it can be seen that the co-efficients are least for filters 4, 5 and 6 indicating thereby that the variations in E are more marked for the low and high or extreme wavelengths than for the medium ones. The reasons for these are probably due to the factors mentioned in the beginning of the discussion. However, it suggests that in order to get more or less constant values of E,

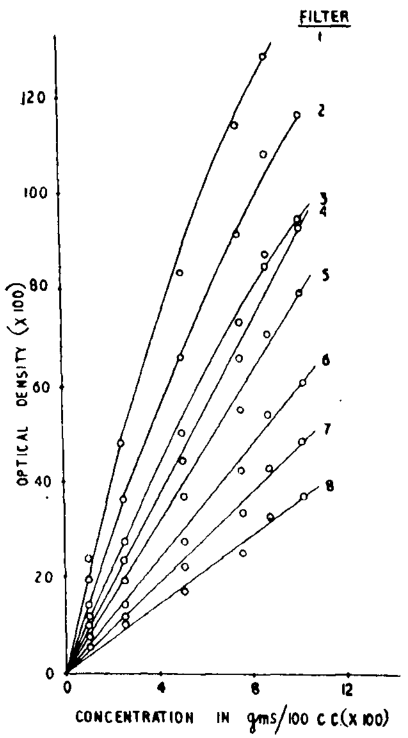
the medium wavelength bands could be used for all practical purposes of identification of bitumens. While it is noted from the Table 4 that there are fluctuations in the values of E (for a given bitumen, solvent and filter), for different values of C, in the graphs connecting D & C (Figs. 3 to 5), the curves for all filters 4-8 are straight lines showing that a direct relationship exists between D & C and that the slope of the line is the constant E for the corresponding filter. For filters 1-3 the curves show a linearity at the low concentrations but a curvature at higher (such as $>.04$) concentrations denoting a dropping off of the value of E at higher concentrations. The reason for this behaviour could perhaps be attributed to one or more of the phenomena expressed in the beginning of this discussion. However, for all practical purposes for the sake of comparisons and identifications, the value of E can be taken to be a constant and equal to the slope of the D & C curves at the origin.



FILTER NO	'E'
1	33·00
2	24·90
3	18·30
4	15·70
5	13·00
6	9·50
7	7·20
8	5·80

Fig. 3. **Curves connecting optical density and concentration for bitumen B-30/40**

Solvent used: Benzene. Thickness of cell: ½ Cm.



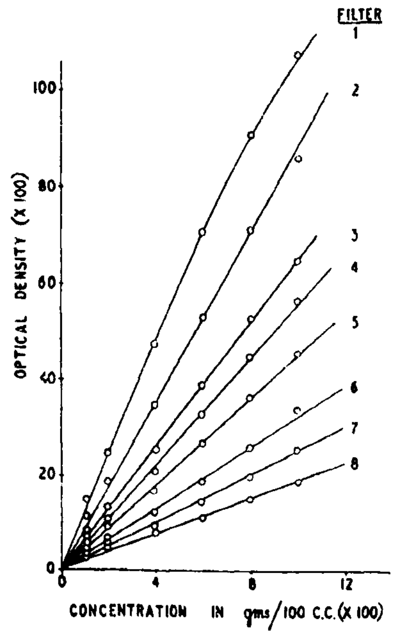
FILTER NO	'E'
1	38·4
2	28·8
3	21·6
4	19·2
5	16·0
6	12·0
7	9·6
8	8·1

Fig 4. **Curves connecting optical density and concentration for blown bitumen B-85/25.**

Solvent used: Benzene. Thickness of cell: ½ Cm.

Whereas in the case of the curves for the penetration grade and blown grade, Figs. 3 & 4, there is a definite curvature in the relationship between *D* & *C* for filters 1-3, in the case of the cutback grade it is not so specially for filters 2 & 3. This is probably because of the fact that the cutback grade is in effect a 80 per cent (approx.) solution of bitumen. So the corrected concentration based on bitumen only would be lower than the concentration shown. Coupled with this, the effect of the solvent oil also might be the reason for the linearity in the relationship.

This brings in a question as to whether for practical considerations, an optimum or critical concentration could be chosen or not. Within the range of concentrations taken in this study,



FILTER NO	'E'
1	24·50
2	18·00
3	13·00
4	11·00
5	9·00
6	6·30
7	5·00
8	4·00

Fig. 5. **Curves connecting optical density and concentration for cutback bitumen R. C. 3**

Solvent used: Benzene. Thickness of cell: ½ Cm.

it can be seen that a linearity of the relationship between *D* & *C* exists for all bitumens for filters 4 to 8. For filters 1 to 3, in all the curves, the relationship between *D* & *C* is linear up to a concentration of 0.04 gms. per 100 cc. In the case of filter 8, Table 4, it is noticed that at a very low concentration such as .01 gms. per 100 cc, the individual values of *E* (from the Table) are slightly higher than the value of *E* from the graph. So it is felt that a range between .02 to .04 gms. per 100 cc. could be termed as the critical or the optimum concentration and could be used quite successfully for obtaining the extinction co-efficient of the bitumen solution directly.

(iii) As a check, a comparison was made to find out the effect of '*l*' with a penetration grade bitumen 80/100 and using the same solvent, namely benzene, a number of tests were carried out at different concentrations for all the filters with two thicknesses of the cell namely $l = \frac{1}{2}$ cm. and $l = 1$ cm. Table 5 gives the values of E (based on curves) for these two cases. A comparison of data in the Table shows that the values of E as obtained from the slope of the D & C curves are more or less the same in the two cases for any given filter. This goes to prove that for all practical purposes the extinction constant can be considered as not being affected by the thickness of the cell.

(iv) In order to check the effect of the nature of the solvent, two series of tests were carried out using a penetration grade bitumen 30/40 with a $\frac{1}{2}$ cm. glass cell with various concentrations and different filters, but using benzene as a solvent in one case and trilene as the solvent in the other case, Table 6. As before, here also the values of E as obtained from the slope of the D & C curves are very close to each other for each filter. This indicates that the nature of the solvent used has no pronounced effect on the values of the extinction co-efficient. But based on other considerations such as light and colour stability, benzene as a solvent is preferred.

(v) Results obtained on the bitumen solution using white light without any filters show that the laws of absorption are not strictly followed.

B. Comparison of Bitumens

Based on the E values as obtained from the slope of the D & C curves and as given in Table 7, the following observations could be made :

(i) An analysis of the average E values, Table 3, indicates that there is no direct relationship between penetration and the extinction constant. Taking the various penetration grades from the same firm, namely B-10/20, B-30/40 and B-180/200 it is noticed that they all have the same E values for any particular filter, although their penetration values are widely different. Again, the E values of B-80/100 are much lower than the correspondiug values for B-30/40 or B-180/200. Similarly, the E values of B-20/30 are quite different.

The E values of A-20/30 are the same as the corresponding E values of S-80/100 and those of A-60/70 are similar to those of the cutback RC 3. Further, the E values of the blown grade B-85/25 are the highest among the bitumens tested in this series, although its penetration value should be in between that of bitumen 10/20 and bitumen 30/40.

In the same way it can be seen that the softening point of the bitumens has no bearing on the E values. Although it can be said that within a group of penetration grades of bitumen from the same firm the penetration values decreased in a way with an increase in the softening point, it cannot be said that the E values are in any way related to the softening point. For example, B-10/20 has a softening point of about 70°C and the B-180/200 has a softening point of about 40°C. But their E values are more or less alike.

As regards ductility also it is observed that it has no bearing either on the optical density 'D' or on the extinction constant E of the bitumen solution. The blown grade 85/25 has the lowest ductility and it has the highest values of E. The Assam bitumen A-20/30 and A-60/70 have also low ductility—owing to their paraffinic crude and owing to the process of air blowing. A-20/30 has the same E values as the S-80/100 which has a higher ductility value. A-60/70 and B-RC3 have similar E values though the latter is more or less a viscous liquid. B-10/20 or B-180/200 have very different ductility values but their absorption characteristics are similar.

(ii) The above discussion very clearly indicates that there is no well defined relationship between the absorption characteristics and the three physical properties of the bitumen. But the variations noticeable in the E values of the various grades of the bitumen imply that the absorption characteristics could be related to the optical characteristics of the constituents of the bitumen. Hence, it is felt that the nature and the chemical composition of the constituents would influence the E values. It is well known that the structure of the bitumen depends on two factors, namely, (a) the source of the crude such as from Assam or Middle East etc., and (b) the method of manufacture such as steam refined, blown etc. Since in the road making industry, the different grades of bitumen are usually identified by the penetration values which can be said in a way to be influenced by the nature and percentage of one or more of its constituents, in this analysis—for the sake of comparison—the E values of the various bitumens would be studied mainly from the stand point of penetration and the supplier.

In this investigation the various bitumens were obtained from three firms. As far as the bitumen from Assam Oil Company is concerned, it can be said that the source is the same and the products were manufactured in the same plant. As regards the bitumens from M/S. Burmah Shell and Standard Vacuum, the various grades might have been obtained from various sources in the Middle East and the products refined at different plants. However, it is attempted to study the E

values with reference to the probable source and method of production.

While going through the E values of the different grades supplied by M/S. Burmah Shell, it is seen that, whereas grades B-85/25; B-RC3; B-20/30 and B-80/100 have distinctly different E values, the other grades B-10/20; B-30/40 and B-180/200 all have the same set of E values. Similarly, regarding the grades obtained from Assam Oil Company, whereas A-30/40 and A-60/70 have the same optical characteristics, A-20/30 is different. In the case of the two bitumens obtained from Standard Vacuum Oil Co., the grade S-80/100 is quite different from S-30/40.

On comparing the absorptive characteristics of the bitumens having the same penetration (irrespective of the supplier) it is noted that the two 80/100 grades, namely B-80/100 and S-80/100, have different sets of E values; the two 20/30 grades namely A-20/30, and B-20/30 have the same set of E-values; and out of the three 30/40 grades, B-30/40 and S-30/40 have similar absorption characteristics whereas A-30/40 behaves differently.

Analysing the data in detail, it is seen that the observations could be divided into the following four categories, namely:

(i) Bitumens having the same set of E values and having the same penetration values such as S-30/40 and B-30/40 or A-20/30 and B-20/30.

(ii) Bitumens having different set of E values but having the same penetration values such as S-80/100 and B-80/100 or A-30/40 and B-30/40.

(iii) Bitumens from the same supplier having same set of E values and having different penetration values such as B-10/20 and B-30/40 or A-30/40 and A-60/70.

(iv) Bitumens from different suppliers having the same E values and having different penetration values such as B-20/30 and S-80/100 or S-30/40 and B-180/200.

In order to understand these seemingly anomalous data, Table 8 is prepared. The various binders analysed are divided into 5 groups, each group having one definite set of absorptive characteristics (or one set of E values) and the groups are numbered in the order of their optical densities, Group 1 being the densest and having the highest set of E values and Group 5 being the least dense and having the lowest set of E values.

From this Table 8 it is seen that the blown grade B-85/25 stands quite distinctly apart and is optically the most dense. This implies that the structure of this bitumen is quite different from that of the rest. Either the source of the crude and/or the method of manufacture is different. The latter is more probable since the blown bitumens are usually manufactured by a special process of blowing air through hot bitumen, bringing about a certain degree of oxidation with a resultant change in the composition of the material.

Optically, the second densest are the grades B-10/20; B-30/40; B-180/200 and S-30/40. From the point of view of penetration and softening point, B-30/40 and S-30/40 are similar whereas B-10/20, B-30/40 and B-180/200 are different. But the absorptive characteristics of all of them are the same. Now, if the source of the bitumens were different for each of the above grades, it means that their optical characteristics are alike either due to the sameness of the crude or due to the differences in the manufacturing processes which have yielded bitumens of different consistencies but having the same E values. If on the other hand the sources of the bitumen were the same, whereas for the grades B-30/40 and S-30/40, the method of manufacture should have been similar, for the grades B-10/20, B-30/40 and B-180/200 slight modifications would have been necessary for obtaining the different consistencies. These modifications are likely to have brought about only such a change in the composition of the bitumen as not to have exerted any noticeable influence on the overall absorptive characteristics of the bitumens as a whole. A more detailed absorptiometric analysis on the constituents of the bitumens would throw some more light on these aspects.

Third in the order of optical density come the bitumens A-20/30, B-20/30 and S-80/100. Here, whereas A-20/30 is definitely of Assam crude, the other two are not. Also A-20/30 is obtained by a process different from that of S-80/100. Based on the physical properties, the three behave quite differently from one another (vide Table 3). This clearly indicates that even if the sources are different and the physical properties are also different, the absorptive characteristics could be similar as a result of the method of manufacture, which seem to have a greater effect on the structure of the bitumens than the source.

A-30/40 and A-60/70 are obtained from the same source as A-20/30. Yet, these two together with B-RC-3 fall in the fourth group. The physical properties of A-30/40 and A-60/70 are different from one another but totally different from A-20/30. The bitumens have been manufactured by resorting to different degrees of air blowing to get the different consistencies. Obviously A-20/30 has been treated to a greater extent than A-30/40 or A-60/70, with the result that it has quite a different structure (absorptiometrically) from the other two. Thus, here also the effect of the method of manufacture on the E values is noticeable.

In this connection it is quite interesting to note that the cutback grade B-RC-3 falls in this group (4th in order of optical density). This grade of bitumen is manufactured by cutting back a parent bitumen with a solvent. The lighter optical density of the cutback could be due to the absorptive characteristics of the parent bitumen itself or it could also be due to the effect of the cutback solvent used at the time of production. Here also the influence of the manufacturing process in the E values is indicated. Some more investigations upon the influence of the different qualities and quantities of the bitumen in the cutback should throw some more light on this aspect.

B-80/100 is optically the least dense of all. This very clearly indicates that the source of the bitumen and/or the method of manufacture of this grade is quite different from the rest.

All these observations go to indicate that bitumens from the same source need not necessarily have the same light absorptive characteristics. Similarly, bitumens of the same penetration grades also need not necessarily have similar set of E values. Similarity in the set of E values could be obtained both for bitumens from different sources as well as for bitumens of different penetration grades as a result of the influence of the method of manufacture used. Diversity in the E values could be either due to the nature of the origin or the source of the crude or due to the manufacturing process employed. So it can reasonably be concluded that the absorptive characteristics are related only to the nature and the composition of the constituents of the bitumen (or rather the structure of the material) as influenced by the source and/or the process of manufacture.

7. CONCLUSIONS

1. The extinction constant E (based on the slope at the origin of the D & C curve) of the bitumen can be used as the

distinguishing factor, and for all practical purposes it could be taken as a constant for a given filter.

2. An optimum or critical concentration of bitumen .02 to .04 gms. per 100 cc. is suggested for obtaining the E values.

3. Colour filters having medium wavelengths such as filters from 4 to 7 are preferable for purposes of comparison.

4. Neither the nature of the solvent used nor the thickness of the glass cell has any pronounced effect on the extinction constant. Only the wavelength of the filters influences it considerably.

5. There is no direct relationship between the various physical properties and the extinction constants of the bitumens.

6. Though, indirectly, the physical properties may be related to the nature of the composition of the constituents of the bitumens, it appears that the absorptiometric characteristics have a closer bearing to the latter than the former. Further investigation on the absorption characteristics of the individual constituents might throw some light.

7. The absorption characteristics are influenced by the oxidation processes and so it can be concluded that based on this the changes in the bitumens as a result of overheating, weathering etc. can easily be detected.

8. The extinction constants are very much influenced by the source and the method of manufacture employed. Indications are that the latter has a more pronounced effect than the former.

9. Finally, it can be said that the absorptiometer can be employed very usefully to a limited extent for purposes of classification and identification of asphaltic bitumens.

8. ACKNOWLEDGMENT

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TABLE 1

Filters

Filter No.	Colour of the filter	Principal spectrum line $m\mu$	Range of over 10 per cent transmission $m\mu$
1	Violet	435	405—455
2	Blue	465	460—470
3	Blue Green	480	475—485
4	Green	520	None
5	Yellow Green	550	None
6	Yellow	575	None
7	Orange	600	590—620
8	Red	$>$675	$>$640

TABLE 3

Physical properties of the bitumen

Sl. No.	Bitumen	Standard penetration 100th of a Cm.	Standard ductility Cm.	Standard softening point °C.
1	B-10/20	11	9.5	70
2	B-20/30	21	32.3	65.5
3	A-20/30	22	9.6	59
4	B-30/40	28	21.5	66
5	A-30/40	28	27.3	56.5
6	S-30/40	38	82.3	58.5
7	A-60/70	52	43.3	55
8	B-80/100	78	Over 100	48.5
9	S-80/100	80	Over 100	49
10	B-180/200	156	Over 100	40
11	Blown bitumen B-85/25	21	4.5	85

(Table 2 overleaf)

TABLE 2

Bitumen chosen for the study

Sl. No.	Nature of bitumen	Name of the supplying company: Burmah Shell (denoted by B)	Standard Vacuum Oil Company (denoted by S)	Assam Oil Co. (denoted by A)
1	Penetration Grade 10/20	B	—	—
2	Penetration Grade 20/30	B	—	A
3	Penetration Grade 30/40	B	S	A
4	Penetration Grade 60/70	—	—	A
5	Penetration Grade 80/100	B	S	—
6	Penetration Grade 180/200	B	—	—
7	Blown Bitumen 85/25	B	—	—
8	Cutback R. C. 3	B	—	—

Total number of grades - 12

N.B.: Burmah Shell bitumen is denoted by the prefix B, for example B-10/20.
Standard Vacuum bitumen is denoted by the prefix S, for example S-80/100.
Assam Oil Co. bitumen is denoted by the prefix A, for example A-20/30.

(*Table 3 on prepage*)

TABLE 4

A typical data sheet giving the values of optical density '*D*' and extinction constant '*E*' for different concentrations

Bitumen. B-30/40.

Solvent: Benzene

Thickness of cell: ½ cm.

Filters	.01 per cent		.0208 per cent		.0392 per cent		.0596 per cent		.0724 per cent		.0908 per cent		.1192 per cent		Coefficient of variation per cent	'*E*' calculated from *D* & *C* curve
	D	*E*	*D*	*E*	*D*	*E*	*D*	*E*	*D*	*E*	*D*	*E*	*D*	*E*		
1	.171	34.2	.339	32.58	.638	32.56	.862	28.92	.968	26.76	1.152	25.38	1.249	20.94	16.71	33.0
2	.127	25.4	.254	24.42	.489	24.94	.687	23.06	.788	21.67	.936	20.60	1.02	17.114	13.15	24.9
3	.095	19.0	.192	18.46	.370	18.446	.52	17.452	.606	16.742	.745	16.412	.885	14.846	8.33	18.3
4	.079	15.8	.163	15.672	.323	16.478	.464	15.908	.563	15.532	.714	15.726	.937	15.718	1.905	15.7
5	.065	13.0	.134	12.884	.266	13.570	.393	13.190	.469	12.958	.593	13.062	.807	13.540	1.922	13.0
6	.048	9.6	.095	9.132	.194	9.898	.282	9.464	.339	9.364	.426	9.382	.594	9.786	2.373	9.5
7	.039	7.8	.076	7.308	.150	7.652	.220	7.384	.264	7.294	.331	7.290	.462	7.756	3.228	7.2
8	.036	7.2	.064	6.154	.121	6.174	.171	5.638	.200	5.526	.247	5.440	.348	5.838	10.048	5.8
White light	.078	15.6	.14	13.460	.258	13.164	.367	12.318	.428	11.824	.520	11.754	.683	11.458	11.158	—

TABLE 5

Comparison of 'E' values for bitumen B-80/100 for different cell thicknesses

Solvent: Benzene

Filter	'E' $l=1$ cm.	'E' $l=\frac{1}{2}$ cm.
1	23.6	23.0
2	15.2	15.5
3	10.2	10.8
4	8.0	7.9
5	6.5	6.4
6	4.5	4.4
7	3.3	3.3
8	2.4	2.4

TABLE 6

Comparison of 'E' values for bitumen B-30/40 for different solvents

Thickness of cell: $\frac{1}{2}$ cm.

Filter	'E' Benzene	'E' Trilene
1	33.00	34.80
2	24.90	25.50
3	18.30	19.00
4	15.70	15.60
5	13.00	13.00
6	9.50	9.20
7	7.20	7.50
8	5.80	5.80

TABLE 7

'E' values (based on the 'D versus C' curve) for the different grades of bitumen

Solvent: Benzene

Thickness of cell: $\frac{1}{2}$ cm.

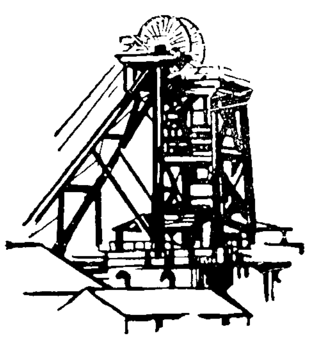
Filter	'E' values B-10/20	B-20/30	A-20/30	B-30/40	A-30/40	S-30/40	A-60/70	B-80/100	S-80/100	B-180/200	Blown B-85/25	B. R. C. 3
1	30.40	30.00	30.00	33.00	26.00	32.00	26.00	23.00	30.60	32.00	38.4	24.50
2	23.00	22.50	22.50	24.90	19.75	25.00	19.70	15.50	23.00	24.00	28.8	18.00
3	17.40	16.50	16.20	18.30	14.00	18.00	14.25	10.80	16.50	18.00	21.6	13.00
4	15.00	14.00	14.00	15.70	11.50	15.00	12.30	7.90	13.00	15.00	19.2	11.00
5	12.60	11.50	11.50	13.00	9.00	12.20	9.75	6.40	10.50	12.00	16.0	9.00
6	9.30	8.25	8.00	9.50	6.50	9.20	7.00	4.40	7.50	9.00	12.0	6.30
7	7.50	6.30	6.00	7.20	5.00	7.00	5.25	3.30	6.00	7.25	9.6	5.00
8	5.70	4.75	4.50	5.80	3.90	5.10	3.70	2.40	4.50	5.50	8.1	4.00

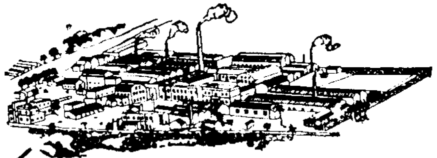
TABLE 8

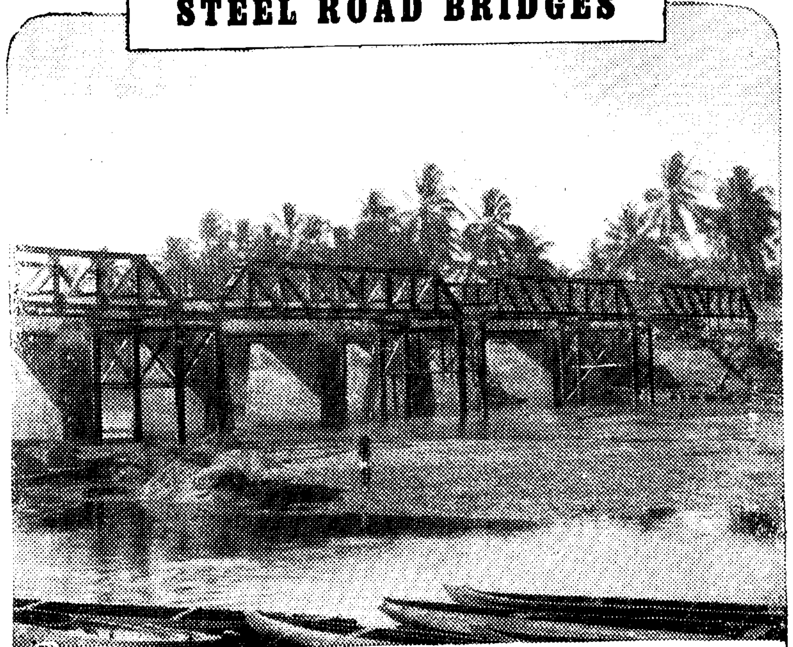
Grouping of bitumens based on 'E' values

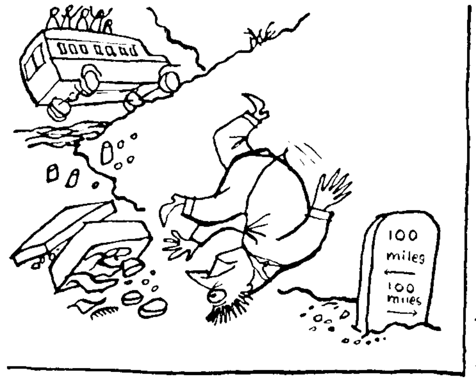
Group 1	Group 2	Group 3	Group 4	Group 5
Blown B-85/25	B-10/20	A-20/30	A-30/40	B-80/100
	B-30/40	B-20/30	A-60/70	
	B-180/200	S-80/100	B.R.C. 3	
	S-30/40			

N.B.: The groups are arranged in the descending order of the values of 'E,' group 1 being the highest.









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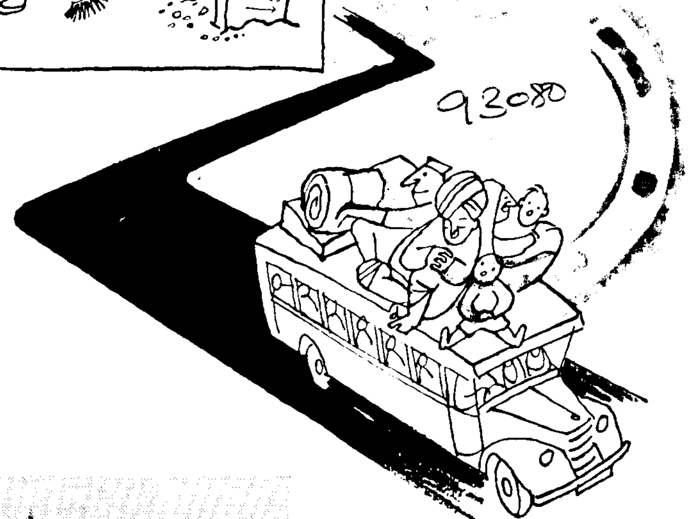

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