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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1082

Journal of the

Indian Roads congress

V. XXIV-1

1959-60

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Chief Engineer, Buildings & Roads, Uttar Pradesh.
Chief Engineer, (Highways), Madras.
Chief Engineer, Buildings & Roads, Punjab.
A representative of Railway Testing & Research Centre, Ministry of Railways, Lucknow.
A representative of Central Standards Office for Railways [Joint Director, Standardisation (Track)].
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P. V. Divatia	Brig. S. K. Bose
R. C. Roy	S. R. Mehra
J. M. Trehan ...	*Member-Secretary*

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-OURTH SESSION OF THE INDIAN ROADS CONGRESS

The next General Session of the Indian Roads Congress ill be held at Bangalore early in 1960. The exact dates id programme will be communicated in due course. Members are quested to contribute Papers on any subject connected with ighway Engineering for discussion at the Session.

2. The Indian Roads Congress will be completing 25 years f life in December 1959. The **Silver Jubilee** of the Congress ill, therefore, be celebrated at the next Session. One of the ays by which this can be done is by presenting for discussion a et of good Papers contributed by members. After all, Papers re an index of the contribution of Engineers to the advancement f the profession and thereby to the progress of the country. At his stage of the country's road development, members will be in avourable position to send suitable Papers embodying the results f their work and experience.

3. **Procedure**: Members willing 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 preferably be submitted as early as possible.**

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 sufficient time to study the Papers well

in advance of the date of the Session, members are particularly requested to **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 the medals are published on pages 10 to 15.

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 consider 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, 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 sujects 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 $2\frac{1}{2}$ 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 discussson 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 up to three months and two annas a day thereafter up to 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 office of the Consulting Engineer (Road Developments) 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.

V. NEW ADMISSIONS

LIST OF MEMBERS OF THE INDIAN ROADS CONGRESS ENROLLED FROM 11-5-59 TO 30-9-59.

Serial No.	*Roll No.*	*Name*	*Address*
1	2584	S. P. Krishna Moorthy	Special Assistant Engineer, (H), Madurai, Panagal Buildings, **Madurai**, (Madras State).
2	2585	Dwarika Singh	Assistant Engineer, I/C. Project Sub-Division, P.W.D., **Chapra**, (Bihar).
3	2586	H. H. Pancholi	Deputy Engineer, P.W.D., (B & R), Govt. Qr. 63/2, **Bhuj**, (Kutch).
4	2587	Sahdeo Prasad	Assistant Engineer, P.W.D., Bettiah, **Dist. Champaran**, (Bihar).
5	2588	R. C. Gupta	Overseer, P. W. D., 222, Shambhu Nagar, Shikohabad, **Dist. Mainpuri**, (U.P.).
6	2589	C. Mozumdar	Assistant Engineer, P.O. Agartala, Shib Nagar, **Tripura.**
7	2590	R. C. P. Sinha	Assistant Trust Engineer, Improvement Trust, Mohalla Ramdhanpur, **Gaya** (Bihar).
8	2591	Mohd. Yousuf Buch	Assistant Engineer, **Poonch** (Kashmir).
9	2592	K. S. Wali	Deputy Engineer, Hubli B & R Sub-Division, Hubli, **Dist. Dharwar**, (Mysore State).
10	2593	S. P. Khullar	Assistant Engineer, **Udhampur** (Kashmir).
11	2594	Girish Kumar	Sub Divisional Officer, P. W. D., (B & R), C/o Executive Engineer, P.W.D., (B & R), Daltonganj Division, **Daltonganj** (Bihar).
12	2595	N. K. Sinha	Assistant Engineer, Special Community Project, P.O. Manoharpur, **Dist. Singhbhum**, (Bihar).

Serial No.	Roll No.	Name	Address
13	2596	A. K. Ansari	Assistant Engineer, P.W.D., (B & R), Sub Division, Geedam, **Dist. Bastar** (M.P.)
14	2597	A. U. R. Somayajulu	Junior Engineer, (Designs), Office of the Chief Engineer (Highways), Begumpet, **Hyderabad-16** (Andhra Pradesh).
15	2598	K. S. Imandar	Executive Engineer, Madhya Pradesh P. W. D., Construction Division No. 2, **Raipur** (M.P.)
16	2599	A. G. Pol	C/o Shri G. Y. Pol, 1008, Raviwar Peth, Bhioigalli, Kolhapur, **District Kolhapur.**
17	2600	G. K. Shinde	Assistant Engineer, P.W.D., (B & R), Garoth Sub Division, Garoth, **Dist. Mandsaur** (M.P.)
18	2601	N. B. Prasad	Assistant Engineer, P.W.D., (B & R), Patna Secretariat, **Patna-1.**
19	2602	M. P. Thakur	Sub Divisional Officer, P.W.D., P.O./at Begusarai, **Dist. Monghyr**, (Bihar).
20	2603	J. K. Shrivastava	Assistant Engineer, P.W.D., B & R, Bijawar, **Dist. Chhatarpur** (M.P.).
21	2604	S. K. Singh	Assistant Engineer, P.W.D., Garhwa, **Dist. Palamu** (Bihar).
22	2605	K. M. Chatterjee	Assistant Engineer, Raghunathganj Construction Sub Division, Development (Roads) Department, P.O. Raghunathganj, **Dist. Murshidabad**, (West Bengal).
23	2606	G. T. Varma	Assistant Engineer, P.W.D., (B & R), **Khargone** (M.P.)
24	2607	Pulakanand Talukder	Sub Divisional Officer, P.W.D., Bharahi Bridge Construction Sub-Division, No. II, **P.O. Lokra** (Assam).
25	2608	R. L. Saksena	Executive Engineer, P.W.D., Dhar Division, **Dhar** (M.P.)
26	2609	T. Thirumalai	Junior Engineer, Highways, 7 Second Cross Road, Raja Annamalaipuram **Madras-28.**

Serial No.	Roll No.	Name	Address
27	2610	V. K. Srivastava	Apprentice Engineer, U.P., P.W.D., B & R Branch, "Aalok", Wazir Hasan Road, **Lucknow** (U.P.).
28	2611	R. N. Jha	Asst. Engineer, P.W.D., B & R, I/C. P.W.D. Sub Division, Dehri, P.O. Dehri-On-Sone, **District Shahabad** (Bihar).
29	2612	A. K. Sharan	Assistant Engineer, P.W.D., B & R, Sharan Bhavan, Babubazaar, **Arrah** (Bihar).
30	2613	R. A. Singh	Sub Divisional Officer, P.W.D., S.C. Survey Sub Division, Aijal, **Mizo District** (Assam).
31	2614	Ch. V. Suryanarayana Murthy	Assistant Engineer, Central P.W.D., Uchapind, **Dist. Khatua**, (J. & K. State).
32	2615	H. D. Matange	Assistant Engineer, M/s. Gammon India Private Ltd., Hamilton House, Ballard Estate, **Bombay-1.**
33	2616	H. S. Joglekar	Engineer, 'Laxmi Niwas', Deshpande Nagar, **Hubli** (S. Rly.).
34	2617	A. V. Ramanujam	Assistant Professor in Civil Engg., M. B. M. Engineering College, **Jodhpur** (Rajasthan).
35	2618	V. N. Chandrasekharan	Special Junior Engineer, Highways, **Kallakkurichi**, (Madras State).
36	2619	K. Venkatanarayana	Assistant Engineer, Highways, Tirupati, **Chittoor District** (A.P.).
37	2623	S. K. Ray	Sub Divisional Officer, P.W.D., Mechanical Sub Division, P. O. **Jorhat, Dist. Sibsagar** (Assam).
38	2621	K. C. Mital	Executive Engineer, U.P., P.W.D., Chief Engineer's Office, U.P., P.W.D., **Lucknow.**
39	2622	M. P. Agrawal	Assistant Engineer, P.W.D., B & R, 118/537, Kaushalpuri, **Kanpur**, (U.P.).
40	2623	P. J. Madan	Professor of Applied Mechanics and Head of the Applied Mechanics Deptt., 14, Alkapuri, **Baroda** (Bombay State).

Serial No.	Roll No.	Name	Address
41	2624	P. H. Gokani	C/o. M/s. Haridas Vithaldas Contractor & Sons, **Dwarka** (W. Rly.).
42	2625	K. Narasimha Chary	Assistant Professor of Civil Engineering, M.B.M. Engineering College, **Jodhpur** (Rajasthan).
43	2626	M. Bhavanarayana-Charyulu	Assistant Engineer, Highways, **Anantapur** (Andhra Pradesh).

ASSOCIATE MEMBER

1	81	M/s. Sahu Cement Service	P.N.B. House, 3rd Floor, 5, Parliament Street, **New Delhi-1.**

VI. DEATHS

The Council have received with regret intimations of the following deaths:

Serial No.	Roll No.	Name	Address
1	1451	S. K. Subramaniam	Managing Director, Gannon Dunkerly & Co., 14, Boag Road, **Madras-17.**
2	161	B. C. Das	Executive Engineer, Division No. 1, Assam P.W.D., **Agartala.**
3	1726	M. Krishnamurti	Executive Engineer, N. E. F. Agency, **Tuensang**, Naga Hills (Assam).
4	60	B. D. Nanda	Divisional Engineer, Retired, Rammunshi Bagh, **Srinagar.**
5	1245	H. E. Ormerod	Upper Burrells, East, Chintington, Lewis, **Sussex**, England.
6	67	Teja Singh Malik	Retired Minister of Development, Patiala Govt., 4, Bhagwan Dass Road, **New Delhi.**
7	1449	J. P. Singh	Superintending Engineer, Nabha Circle, **Nabha.**
8	1923	J. C. Gauba	Trust Engineer, D. 58/12. Sigra, **Varanasi-1.**
9	1804	G. N. Senapathy	Executive Engineer, Thoriasahi, **P.O. & District Cuttack** (Orissa).

VII. ADDRESSES WANTED

Communications addressed to the following Members have been returned unclaimed by the Post Office. Members able to give information regarding the present address of any of them are requested to forward particulars to Secretary, Indian Roads Congress.

Serial No.	*Roll No.*	*Name*	*Address*
1	2113	Adhiraj, T.	Assistant Instructor, Engineerin College, Guindy, **Madras-25.**
2	652	Guruswamy, S.	No. 8, Appaswami Koil Street, P.O. Mylapore, **Madras.**
3	1796	Nagarajan, R.	C/o. Shri K. S. Radha Krishnan, 'Deva Nivas,' Thiruvanai Koil P.O., **Trichy District** (Madras State).
4	183	Nawab Ahsan Yar Jung Bahadur	Afsar Manzil, Jubilee Hill, **Hyderabad-Deccan.**
5	417	Parang, Hari Ram	Retired Executive Engineer, Lal Chowk, Qila Gujar Singh, **Lahore** (Pakistan).
6	1836	Sabharwal, M. L.	1, Station Road, **Kotah Junction** (Rajasthan).
7	2076	Shaikh Mahboob	Executive Engineer, P.W.D., Osmanabad, **Dist. Osmanabad.**
8	1795	Subbayya, K. B.	2108, College Apartment, Uttah State Agricultural College, Logan, **Uttah,** U.S.A.
9	1835	Veerappan, N.	Junior Engineer, **Kilvathnamkuppam,** (N. A. District), (Madras State).

Paper No. 213

"SOME STUDIES IN SOIL MECHANICS AS APPLIED TO ROADS IN WEST BENGAL"

By

S. N. GUPTA,* B.E., M.S. (U.S.A.), M.A.S.C.E., M.I.E. (IND.)

AND

P. L. DE,† B.SC. (HONS.), M.SC.

CONTENTS

SYNOPSIS

The Paper describes some applications of soil mechanics in the construction and maintenance of roads. The general properties of soil and the methods of soil survey and sampling are described. Examples of application of scientific studies of soil in different stages of road construction, viz., construction of embankments and cuttings, design of the pavement thickness, improvement of poor subgrades and different methods of soil stabilisation as adopted in West Bengal are given. Investigation of road failures and their suggested remedies are also dealt with in the Paper.

* Chief Engineer, Construction Board, Government of West Bengal.

† Assistant Director, Roads and Buildings Research Institute, Government of West Bengal.

1. INTRODUCTION

1.1. Of all the ancient arts, road-making is the latest to have called for the aid of laboratory scientists. With launching of gigantic development programmes in the country in different spheres, came the necessity for the construction of many miles of modern roads within the shortest period of time and within the limited resources available. The enormous growth in motor transport which now constitutes the most important form of traffic has focussed attention on the serious problem of constructing and maintaining roads cheaply but at the same time strong enough to carry the traffic and adequate in width and layout to permit it to travel safely at reasonable speeds.

1.2. Growth of Soil Mechanics in the Field of Roads

The origin of what is termed "Soil Mechanics" dates from the work of Coulomb nearly two hundred years ago. However, until the beginning of the present century the subject was concerned mainly with the stability of earthworks. Subsequently the subject has developed to cover the design of building and road foundations and other aspects of road construction. The principle functions of soil mechanics in road construction are to assist the Engineer in (i) deciding the thickness and type of pavement required for known traffic and soil conditions and (ii) improving where possible the condition of the soil so that a more durable and possibly less expensive pavement can be constructed.

2. GENERAL PROPERTIES OF SOIL

2.1. Soil Formation and Definition of Terms

2.1.1. The material of which the crust of the earth is composed can be broadly classified into two groups, viz., rock and soil. The soil has been formed from the parent rock by two main processes, viz., (a) physical disintegration and (b) chemical decomposition, their combined action being known as 'weathering'. After formation soil may still be located at the place where it originated from or may be transported and deposited elsewhere on the top of the bed rock. The former is called 'residual soil' and the latter 'transported soil'. The principal constituents of soil are (i) sands and (ii) silts, which are inert fragments of the parent rock, and (iii) clays containing active minerals formed by chemical changes in the rock. Besides these soil contains varying proportions of salts, organic matter, etc.

2.1.2. Methods of classifying soil in terms of their predominant particle groups (sand, silt and clay) tend to concentrate attention on the solid mineral particles which make up the soil. It is, however, important to remember that soils also contain water and air, and whilst the particle size distribution defines the "type" of soil, these latter factors are most important in defining its "state" on which the mechanical properties largely depend.

2.1.3. Depending on the percentage of water present in a soil, it may remain in different 'states', viz., solid, semi-solid, plastic and liquid states and the particular moisture contents of a soil at which it passes from one state to another are important values which are dependent on the type of soil and help in the classification and identification of soil. These values are termed as shrinkage limit, plastic limit and liquid limit. The difference between liquid limit and plastic limit is termed "plasticity index" and signifies the range of moisture content within which the soil remains in the plastic state.

2.2. Classification and Identification of Soil

2.2.1. A suitable method of classifying the soils, with which the construction engineer may have to deal, into different groups according to their relative suitability for use in roads and simple methods of identifying them are essential prerequisites for any road construction work. There are several methods of classification of soil, each method having relative suitability in the fields of Agriculture, Geology, Ceramics, Foundation Engineering, Roads, Irrigation, etc. Again in the same field different methods of classification are adopted in different countries and even in different States and departments in the same country. This sometimes leads to confusion and misunderstanding of the results from one organisation to another. There have been several attempts to adopt a single system of classification in India, at least in the field of engineering, but the problem still remains unsettled as the supporters of one particular system do not like to change over to another system.

2.2.2. Highway engineers generally prefer to adopt the Public Roads Administration (P.R.A.) system of classification (1945) (vide Appendix 1) for the following reasons :

(i) The system is the simplest one and involves minimum number of tests required to be done.

(ii) The system directly tells about the relative suitabsility of a particular soil for use in roads and requireminimum number of terms and factors to be remembered.

(iii) The system with some modifications[1] covers almost all types of soils available in India.

(iv) The Group Index number enables a systematic graphical representation of the various soils met with in different depths in a bore or at different places along an alignment and helps in selecting suitable soil for embankment, Plate I.

2.2.3. The next preferred system of classification is the "Casagrande system". This system defines the different types of soil in a more comprehensive way and has better applicability in fields other than roads, viz., Foundation Engineering, Irrigation, Embankments, etc. The Roads and Buildings Research Institute of West Bengal has been, for some time, adopting both the P.R.A. and Casagrande systems of classification in its routine test reports, but experience has shown that the former is better understood and preferred by the field Engineers.

2.2.4. The E.C.A.F.E. Seminar on "Low-cost Roads and Soil Stabilisation"[2] held in New Delhi in January, 1958 has recommended the adoption of "Unified Soil Classification System", which is a revision of the Casagrande system, with some further modifications for Indian soils, Appendix 3. The Indian Standards Institution (I.S.I.) has finalised the details of this Unified Soil Classification system on the advice of the C.R.R.I.

2.2.5. **Field Identification of Soil**

The main groups into which the soil can be easily classified by simple field tests without the use of any equipment are coarse sand, medium sand, fine sand, silty soil, clayey soil and peat or organic soil. An experienced person can further identify the soil into different sub-groups.

2.2.6. The correct method of identification of the different types of sand is by sieve analysis. This will, however, necessitate the carrying of standard sieves, balances, etc., and may not, therefore, be convenient in preliminary survey work. A simple method is to carry small samples of sand of the different types and to compare with them the material encountered in the field by visual examination.

2.2.7. There may be some confusion in distinguishing silty soil and clayey soil by visual examination alone, but there are some simple field tests for distinguishing them, Appendix 2.

2.3. Besides the type of soil, the 'state' in which it exists has a great influence on the mechanical properties. This state is indicated by the amount of water and/or air present in the soil mass and is mathematically represented by its dry density. The denser the soil mass the stabler and stronger it is for road foundation and embankments.

3. SOIL SURVEY AND SAMPLING FOR ROADS

3.1. Purpose and Scope of Soil Survey

3.1.1. Soil survey is an essential part of preliminary engineering survey for roads and it should receive due importance in any important road project. The information obtained from such soil survey will make available to the Engineer useful data for deciding about the alignment of the road and for an economical and at the same time safe design of the pavement thickness.

3.1.2. A soil survey for road purposes mainly involves two distinct operations, viz:

(a) The exploration of the soil upto a certain depth by boring and collection of boring samples.

and

(b) The examination and testing of the boring samples and preparation of soil profile.

Besides the above two major operations which should cover the entire alignment of the road at as close distance intervals as possible, the soil survey also includes:

(i) Collection of undisturbed samples from area of weak soil formation and evaluation of the safe bearing capacity from laboratory tests as well as site plate load or C.B.R. tests.

(ii) Search for better type of soil or granular material where necessary, at economic distance, including sampling and testing.

(iii) Investigation of ground water and waterlogging conditions.

3.2. Soil Exploration by Boring

3.2.1. The purpose of soil exploration is to show a vertical section indicating the nature of soil at different strata over which the road alignment passes. This information will help in

(i) determining the suitability of the soil at different depths for constructing the embankment and for that purpose the depth upto which borrow pits should be made and also in (ii) designing the slope at which cuttings and embankments will stand.

3.2.2. The Roads and Buildings Research Institute of West Bengal undertook soil survey for the proposed "Calcutta-Durgapur Expressway." The first phase of the work involved (a) collection and testing of boring samples upto 10 ft depth in the location of borrowpits along the proposed alignment, (b) conducting density and C.B.R. tests on undisturbed samples of subgrade soil from the already constructed portion of National Highway No. 2, (c) conducting field plate load tests on selected spots on the above highway.

3.2.3. The types of soil met within a few typical borings are shown graphically in Plate I. The boring results show that the type of soil changes at different depths in different locations and from a study of the curves it is possible to determine the depth of borrow pits upto which the soil should be used in constructing the embankment.

3.2.4. The idea of conducting density and C.B.R. tests on the subgrade soil of the already constructed portion (Mile $3\frac{1}{2}$ to Mile $9\frac{1}{2}$) of N. H. 2 was to ascertain the degree of compaction and bearing capacity of the subgrade in order to design the additional thickness of hard crust required to raise this portion of the N. H. 2 to the standard of an Expressway.

3.3. Investigation of Ground Water and Waterlogging Conditions

3.3.1. Ground water plays a very important part in the stability of a road. It is well known that the bearing capacity of soil changes to a great extent with the change of moisture content. This is more so in the case of clayey soils. The design of flexible pavement thickness is generally based on either the in-situ C. B. R. test results or on the results of C. B. R. tests conducted in the laboratory on undisturbed samples of soil after 4 days immersion in water. These results are sometimes misleading as the moisture condition in the natural subgrade may remain quite different from the conditions under which the tests were made. This may lead to either uneconomical design or an unsafe design. Hence a knowledge of the maximum possible moisture content of the subgrade in the worst condition is necessary. The recording of the moisture content of the subgrade at different depths and in different seasons is, therefore, a very useful investigation. This investigation has been undertaken by the Central Road Research Institute in different zones of the country and the Roads and Buildings Research Institute of West Bengal is also co-operating in making some measurements in the State.

3.3.2. Waterlogging conditions caused by irrigation systems may also lower the bearing capacity of the subgrade soil and the chances of such casualties are being taken into consideration while making soil survey for roads.

3.4. Soil Sampling

A knowledge of the correct method of soil sampling and the type and quantity of soil samples required for different purposes is a desirable prerequisite to soil-survey. Soil samples are generally classified into two types, viz., (i) undisturbed and (ii) disturbed.

3.4.1. **Undisturbed samples** of soil are required for conducting certain special tests, viz., (i) shear tests, (ii) consolidation tests, (iii) unconfined compression tests, (iv) natural moisture content and density, (v) California Bearing Ratio tests, etc. While collecting undisturbed samples, necessary precautions must be taken to ensure that all the properties of the earth mass of which it is representative are retained. These properties are: the structure of the soil, its texture, its moisture content and density, the stress conditions in the soil, etc. The size and quantity of samples required for different purposes are shown in Appendix 4.

4. CONSTRUCTION OF EMBANKMENTS AND CUTTINGS

4.1. Embankments

4.1.1. The selection of the right type of material to be used as fill in embankments is very important and requires a thorough knowledge of the fundamental properties of soil. The principle considerations while making a choice should be that the material should have proper grading and low capillarity. The American Association of State Highway Officials recommends [3] that for fills not exceeding 10 ft in height and not made on steep slopes or subject to long inundation, the liquid limit of the soil should not exceed 65 and the plasticity index should not be less than $0.6 \times$ L.L.-9.0. For fills exceeding 10 ft in height or placed on steep slopes or subject to inundation, the liquid limit of the soil should not exceed 50. Granular materials in which at least 65 per cent is retained on a No. 200 sieve can be safely used.

4.1.2. The importance of compacting the embankment to the maximum attainable density can never be over-emphasized.

Mehra and Uppal [4] in a Paper entitled "The Importance of Subgrade Compaction in the Economic Design of Flexible Pavement" have elaborately studied this subject and have shown how a little extra expenditure incurred on the proper and controlled compaction of the fill material results in ultimate economy and durability of the road. A common practice is to allow a new embankment to settle under its own weight during a few monsoons and thereby get consolidated. This method, however, is not fully reliable particularly when there is not sufficient time lag between the construction of the embankment and the laying of the pavement.

4.1.3. The Roads and Buildings Research Institute of West Bengal had occasions to study the cause of failure of several newly constructed roads where this was attributed to inadequate compaction of the fill material. It will be interesting to mention here a few of these examples.

(i) **Balighai — Mohanpur Road** (O.D.R.) in the District of Midnapore.

It was observed that the stone metalled surface (completed in July-August 1958) had been much damaged in stretches for about 2 miles and the subgrade soil had become highly unstable after the monsoon of the same year, though the intensity of traffic was only of the order of 50 vehicles exceeding 3 tons per day. For the purpose of investigation 3 portions were selected, one at 3rd furlong of 1st mile where no damage of the road had taken place and the other two at 7th furlong of 2nd mile and 3rd furlong of 4th mile where considerable damage to the road had occurred. Undisturbed samples of soil were collected in February 1959 from about 6 in. below the subgrade and were tested in the laboratory. The results of the laboratory tests together with the field observations are shown in Table 1.

The results show that the type of soil is more or less similar in all the three portions. However, the density of the natural subgrade in the damaged portions is much lower than that in the undamaged portion and consequently the C.B.R. value is lower and the tendency to absorb moisture is more in the former case. The cause of damage was attributed to inadequate densification of the fill material and it has been recommended to take out the existing crust, loosen the subgrade for a depth of at least 15 inches and compact the same at optimum moisture content so as to get a dry-density of not less than 106 lb per cu. ft. and then relay the hard crust as previously done. If sand be available economically the double brick flat soling may be replaced by 6 to 7 in. thick soil-sand stabilised base.

TABLE 1

Results of investigations on the failure on Balighai—Mohanpur Road (O.D.R.)

Location	Condition of road	Original specification and thickness of hard crust	Existing thickness at the time of observation	Height of embankment	Water table (in February) and H.F.L.	Year of construction of earthwork for embankment	Natural water content of soil at time of sampling% (Feb. '59)	Natural dry density of soil	L.L.	P.I.	C.B.R. tests after saturation for 4 days: C.B.R. value	C.B.R. tests after saturation for 4 days: Moisture content%
3rd Furlong of 1st Mile	No damage	Double brick flat soling 4½ in. (loose) jhama consolidation. 3 in. (loose) stone metal consolidation. Total thickness = about 11½ in.	About 11½ in.	3 ft from field level. 5 ft from borrowpit bed level.	5 ft below road level 1½ ft below road level	1955-56	25.7	1.7 G.M./C. C. 106 lb/cu. ft.	52.8	22.5	5.8	27
7th Furlong of 2nd Mile	Damaged	—do—	About 8½ in	4 ft from field level. 5½ ft from borrowpit bed level.	6 ft below road level 2 ft below road level	1955-56	23.7	1.36 G.M./C. C. 85 lb/cu. ft.	51.1	21.7	1.7	35
3rd Furlong of 4th Mile	Damaged	—do—	About 8½ in.	5 ft from field level. 7 ft from borrowpit bed level.	5½ ft below road level 3 in. above road level.	1956-57	24.4	1.45 G.M./C. C. 90.6 lb/cu.ft.	50.0	29.0	2.0	35

(ii) N.H. 2 to Durgapur Barrage (S.H.) in the District of Burdwan.

The embankment of this newly constructed road had settled in several places during the rainy season of 1958. The embankment is high at some places and had not passed the normal settlement time. The proposed specifications were 6 in. boulder soling with 3 in. moorum filling and 6 in. (loose) stone metal consolidation with one coat surface dressing. However, before the metalling was laid, it was oberved that the boulders used in soling had gone down as much as 2 ft below their formation level under wheel load at some places. This called for an investigation to ascertain whether the proposed thickness of metalling would be adequate under the existing conditions. The subgrade soils from badly damaged portions were tested for C.B.R. and other properties in order to design the required thickness of the pavement. The results are shown in Table 2.

From the results it was observed that the dry density of the subgrade soil was low and its moisture content was higher than the plastic limit and only about 2 per cent less than its moisture content after 4 days complete immersion in water. This showed that the embankment had not properly settled and the C.B.R. values indicated that unless a very thick pavement and base was used, the road may be liable to be damaged due to the gradual consolidation of the subgrade under traffic. Further, the road is on a very high bank which is likely to have considerable amount of voids inside. So the C.B.R. values, as determined on the top layer of the fill only, for designing the pavement thickness would not help much, but rather they would be misleading as when the voids inside the embankment would be filled up by natural process in course of time, the road formation was bound to subside whatever be the pavement thickness and thus damage the road.

It was suggested that the completion of this portion of the road to the designed specifications should be delayed till the embankment had undergone the full natural consolidation. The road in the meantime should be maintained just in a passable condition by providing cheap type of thin crust frequently maintained, if necessary. This would help the early consolidation of the subgrade to a great extent under traffic.

(iii) Section Uttarpara—Kalipur Road to Nabagram (Serampore) of N.H.2.

This portion of the road (about 6 miles) is on a newly constructed embankment and the pavement had recently been

TABLE 2

Results of tests on samples of subgrade soil from N.H. 2 to Durgapur Barrage (S.H.) (Samples collected on 6-9-58)

Location	Natural water content at time of sampling. per cent	Natural dry density	Percentage retained on 200 Mesh per cent	L. L.	P. L.	P.I.	Type of soil	C.B.R. tests after 4 days saturation		Requirement of pavement thickness for traffic density of 150 to 450 vehicles, exceeding 3 tons per day
								C.B.R. value	Moisture content per cent	
5th Furlong of 3rd Mile	15	1.60 G.M./C.C. 100 Lb/cu. ft.	44	31	13	18	$\frac{A_6}{CL}$	4.2	17.5	16 in.
2nd Furlong of 4th Mile	17	1.64 G.M./C.C. 102.5 Lb/cu. ft.	36	36	16.5	19.5	$\frac{A_6}{CI}$	5.7	19	14 in.

completed with the specification of double brick flat soling and 7½ (loose) stone metal consolidation. Some portions of the embankment were constructed by earth moving machinery and were somewhat compacted, though not under controlled conditions, while other portions were constructed by manual labour only and no compaction had been done. It was proposed to improve this road to the standard of an Expressway and investigations were undertaken to study the condition of the embankment and to design the thickness of pavement.

The investigation consisted of:

(a) Determination of the nature of soil and its density at the top of the subgrade and at 3 feet depth at several places covering earthwork both by machinery and manual labour, and compare the results with density attained in the laboratory by the Proctor's compaction and by the A.A.S.H.O. modified compaction.

(b) Conducting laboratory C.B.R. tests on undisturbed subgrade soil.

(c) Conducting C.B.R. tests on remoulded samples of subgrade soil compacted at 95 per cent and 100 per cent Proctor Density.

(d) Conducting Field Plate Load tests on the metalled surface and on the subgrade, at locations where earthwork was done by machinery and by manual labour, to evaluate the modulus of subgrade reaction.

The results in Appendix 5 & Table 3 show that, (i) for similar type of soil, the average density of the embankment was greater where the earthwork had been done by machinery than

TABLE 3

Results of field plate load tests on Section Uttarpara — Kalipur Road to Nabagram of N.H. 2.

Location	Earthwork done by	Value of 'K' on metalled surface	Value of 'K' on subgrade
5 M. 1 F.	Manual labour	222 lb per sq. in.	92 lb per sq. in.
4 M. 8 F.	Machinery	705 lb per sq. in.	109 lb per sq. in.

where it had been done by manual labour and the corresponding C.B.R. values were also higher; (ii) the density attained by earthwork done by machinery is higher than the Proctor's density but lower than the A.A.S.H.O. density while the density attained by manual earthwork is lower than the Proctor's density; (iii) the C.B.R. value of a particular soil is considerably reduced if the density is less; (iv) the modulus of subgrade reaction of a machine compacted subgrade is greater than that of subgrade done by manual labour and the compaction of the subgrade has pronounced influence on the modulus of subgrade reaction of the metalled surface also.

The above examples clearly indicated the importance of compaction of the embankment under properly controlled conditions. It was desirable to compact the embankment in layers of not more than 9 in. at optimum moisture content to get adequate density.

4.2. Cuttings

In cuttings, the side slopes should be designed according to the conditions of stability of the soil. In special cases it would be desirable to determine the shear strength of the soil and design the safe slope. Side slopes should be as steep as stability conditions permit and even vertical sides should be adopted in stable rock formation. To keep the side slopes (both for embankments and cuttings) stable some sort of plantation should be provided particularly in cases when the soil was liable to erosion and guttering.

5. DESIGN OF PAVEMENT THICKNESS

The main functions of a pavement are (i) to provide a suitable wearing surface and (ii) to distribute the wheel load sufficiently so that the pressure reaching the subgrade soil is within its safe bearing capacity. The aim of pavement design is to evaluate the type, thickness and treatment of materials to fulfil the requirements most economically. Flexible pavements generally consist of bitumen or tarmacadam surfacing laid on compacted stone metal or other aggregate over a suitable base consisting of bricks, boulders, gravel or mechanically stabilised soil, etc. Concrete is the only important form of construction under rigid pavement. Cement stabilised soil construction might perhaps be considered as intermediate between rigid and flexible and conveniently called as "semiflexible" pavement.

5.1. Flexible Pavements

5.1.1. The required thickness of a flexible pavement mainly depends upon the strength of the subgrade provided the base and sub-base courses are adequately compacted. The strength of the subgrade depends on its moisture content, density and soil type. There are several methods of pavement thickness design practiced in different countries and by different organisations. These different methods utilise the results of one or more of the standard tests, viz., classification tests (Group Index method), C.B.R. test; North Dacota Cone Bearing value; Plate Bearing tests; Shear strength, etc. The Main Roads Department of Queensland, Australia, has developed a method of flexible pavement thickness design based on the 'Linear Shrinkage' of the subgrade soil which can be very easily determined in the field.

5.1.2. The Roads and Buildings Research Institute of West Bengal had, in the past, been adopting the Group Index method for a rough estimate of the pavement thickness though in actual practice a lesser thickness was adopted, Appendix 6. The Group Index method was, however, soon replaced by the C.B.R. method. A few typical examples of pavement design by these two methods are shown in Appendix 7. Standard reference curves are shown in Fig. 1. The required thickness by the C.B.R. method and by the Group Index method campared in Appendix 7 show that the Group Index method generally gives higher values of thickness. As the Group Index method does not take into consideration the degree of compaction and saturation of the subgrade, the results are sometimes misleading. The C.B.R. method of flexible pavement design is the most widely adopted of the methods put forward in recent years and many Highway Authorities have now switched over to this method.

5.1.3. The C.B.R. test apparatus and method [5] are quite well known but the selection of test condition is most important. The undisturbed subgrade soil must be tested under its worst condition of moisture content as anticipated in the site. As it is very difficult to ascertain correctly the possible degree of saturation for individual cases, it has been recommended, arbitrarily, to immerse the sample, in mould, in water for 4 days before test by which time the soil is expected to reach 100 per cent saturation. This sometimes leads to uneconomical design particularly in regions of low water-table and low rainfall. The Central Road Research Institute has initiated an investigation [6] to record the moisture content of subgrade at different depths and at different seasons throughout the various regions of India (including West Bengal where the records are being taken by the State

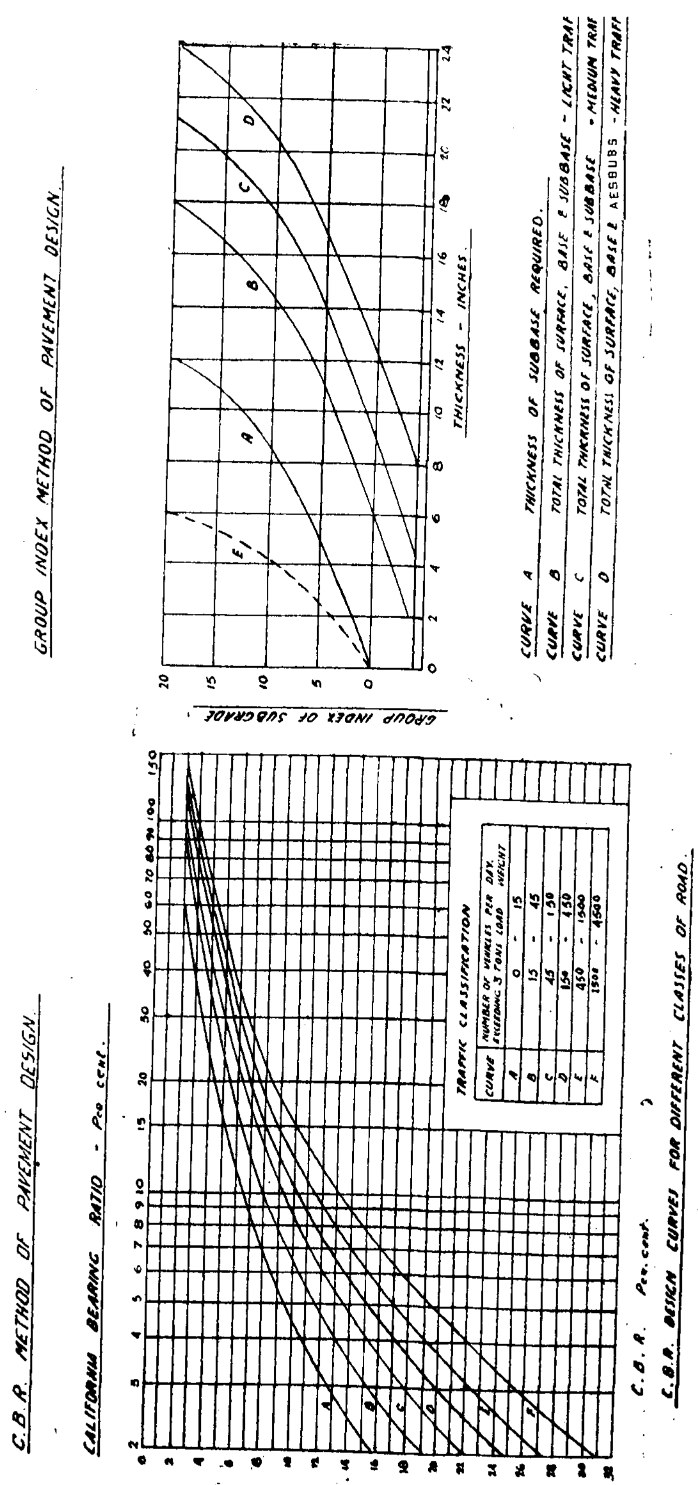

Fig. 1

Roads & Buildings Research Institute) and the findings are expected to provide useful guide to the degree of saturation.

5.2. Rigid Pavement

The structural rigidity of a concrete slab depends on its elastic modulus and the thickness required depends on the properties of the subgrade soil, the intensity of traffic, the amount of reinforcement, if any, and the inclusion or omission of a base. The value of "modulus of subgrade reaction (K)", determined by field plate load tests, is sometimes utilised in the design of thickness of slab. However, the bearing value of the soil has little effect on the stresses due to wheel loads in a concrete slab when it is uniformly supported[7]. Because of this it is considered that tests which measure the strength of the subgrade soil are of limited value in designing concrete road slabs. Evidence shows that under given traffic conditions, the same thickness of slab is suitable for the majority of different types of subgrade. In an experiment[8] made on the Oxton by-pass in Nottinghamshire, where the soil is a gravelly sand, the use of cement stabilised sand base 2 in., 4 in. and 6 in. thick had contributed nothing to the structural performance of concrete slabs upto 6 in. thick.

5.3. Semi-flexible Pavements

5.3.1. Under semi-flexible pavements may be classed two types of constructions, viz., soil-cement and dry-lean concrete both of which possess appreciable flexural strength. These pavements are more efficient than flexible pavements in reducing the load transmitted to the subgrade and can be expected to give equal service with less thickness than required for flexible pavements. It is, however, still too early to express a correct ratio between the thickness required for a flexible and a soil-cement pavement under similar conditions. Martin[9] found that for the particular conditions investigated, the factor by which normal flexible pavement design depth should be multiplied to give the required depth of a soil-cement pavement was about 0.7. Similar evidence[10] has been obtained on roads on two housing estates where 6 in. of soil cement covered with a surface dressing had proved adequate on subgrade having C. B. R. values as low as 7 per cent for which about 10 in. of bituminous-surface construction would usually be recommended for the traffic using these roads. Reductions of 25-33 per cent had often been used and such pavements had given good service. It is, however, felt that the thickness of soil-cement should not be below 6 inches.

5.3.2. Table 4 shows a few examples of soil-cement pavements constructed or proposed to be constructed in West Bengal in which the thicknesses and specifications of the soil-cement had been compared with the thicknesses of flexible pavements that would have been required for these roads. More details of soil-cement roads are given in Chapter 8.

6. IMPROVEMENT OF SUBGRADE

6.1. It is not always possible to obtain a subgrade material which is, in itself, adequate for supporting the loads transmitted to it, neither is it always economical to provide sufficient pavement thickness to disperse the wheel load to such an extent that the actual pressure coming on the subgrade is within its safe bearing capacity. Improvement of the subgrade by artificial means is called for in such cases. Provision of adequate drainage to maintain the subgrade bearing values and adequate compaction of the subgrade to increase the bearing value are the most common means of improvement of the subgrade. The importance of subgrade compaction has already been described in para 4.1.2.

6.2. Several methods [3] of subgrade-surface treatments had been tried with success in other countries. These include Tar-paper covering, oiling, blanket course, etc. Thin 'Alkathene' films are also claimed to be useful as protective covering on subgrades. Practical data is, however, lacking.

6.3. Granular stabilisation of the subgrade, by mixing sand or other locally available coarse material with the top layers of earth filling and then compacting under controlled conditions, is also a means of improving the subgrade. This method had been tried with success on a few roads in West Bengal.

6.3.1. In the Raghunathganj Dhulian Section of N. H. 34 (in the Dist. of Murshidabad) the specification for the pavement was 6 in. boulder soling, 5 in. metalling and $\frac{3}{4}$ in. premix carpetting. However, this was felt to be inadequate as the soil in the subgrade was a bad type of clay (P. I. = 28), fine sand (F. M. = 0.9) was locally available and the top 4 in. of the subgrade in mile 3, 4, 6 to 8 and 11 to 13 was stabilised with this type of sand (in proportion of 1 soil : 2 sand) before laying the boulder soling. By this means the dry density of the subgrade was increased from 91 lb per cu. ft. to 115 lb per cu. ft. The construction was done in 1954 and it had been observed that the portions where the subgrade was thus improved had been behaving well whereas in the untreated portions the

TABLE 4

Pavement thickness of soil cement as compared to ordinary flexible pavements

Name of Road	Nature of subgrade soil: Material passing 200 Mesh per cent	L. L.	P. I.	Natural dry density	C. B. R. value	Moisture content at C.B.R. test per cent	Requirement of flexible pavement thickness as per C.B.R. curves	Actual thickness of soil-cement as adopted or proposed	Reduction for semi-flexible pavement per cent	Remarks
1. Barasat—Basirhat Road (Dist. 24-Pargs.)	96	24	6	99 lb/cu. ft.	5	20	15 in. (curve D)	9 in. soil-cement plus 1½ premix carpet	30	The construction was done in April, 1957 and the pavement has been behaving well. In adjacent portions of the road having normal water-bound macadam specification the thickness adopted is about 12½ in. double brick flat soling, 6 in. loose stone consolidation and 1½ in. premix carpet.
2. Talit—Gushkara Road (Dist. Burdwan)	100	45	25	102 lb/cu. ft.	7.0	26	11 in. (curve C)	6 in. soil-sand cement (1 : 2 plus 8% cement) plus 2 in. premix carpet	27	Construction done in 1957–58.
3. Raghunathganj—Mitrapur Road 4th M. (Dist. Murshidabad)	63	20	6	110 lb/cu. ft.	6	18	12 in. (curve C)	9 in. soil-cement plus 1 in. premix carpet	17	Proposed to be constructed.

boulders had penetrated into the subgrade with consequent undulations of the road surface. The cost of this additional treatment of the subgrade was only about 5 per cent of the cost of the pavement. Similar improvement of the subgrade was also tried, on an experimental basis, on Sagrai-Raina Road (Dist. Burdwan) when the subgrade soil was clayey (P. I. = 32).

6.4. Treatment of clayey subgrades (particularly black cotton soil) with small percentages of lime (about 5 per cent) is also reported [6] to be very effective in improving the subgrade. Laboratory experiments carried out at the Roads and Buildings Research Institute of West Bengal had shown considerable increase in the C. B. R. values of soil treated with small percentages of lime, Table 5.

7. GRANULAR STABILISED (MECHANICALLY STABILISED) SOIL AS A BASE COURSE

7.1. The use of mechanically stabilised soil as a base course for water-bound macadam had now passed the experimental stage and was being widely adopted in several parts of the country including West Bengal [11]. In West Bengal this process had already been tried on more than 20 cases when the conventional double brick flat soling had been substituted by adoption of 7 in. to 9 in. compacted thickness of suitably stabilised soil-sand base course (Examples in Appendix 8). It was estimated that by adoption of this process on large scale construction a saving of Rs 6,000 to Rs 12,000 per mile was possible under conditions prevailing in West Bengal.

7.2. The fundamental principle of granular stabilisation is that by suitably blending the naturally occurring soil with coarse material (commonly locally available sand) and compacting the mix at optimum moisture content a hard layer is produced whose density and C. B. R. values are considerably higher than those of the subgrade as a result of which the required thickness of hard crust to be made up of foreign materials (e.g. bricks, jhama khoa, stone metal etc.) is much reduced. Appendix 9 gives a few examples of the increase in the density and C. B. R. values of subgrade soil as a result of granular stabilisation.

7.3. Besides soil-sand stabilised base courses, a few field experiments on soil-sand-shingle and soil-sand-jhama khoa stabilisation were also conducted. Details of these experiments are given in Appendix 10.

TABLE 5

Showing the increase in C.B.R. values of soil treated with lime

Source from where sample collected	Type of soil: Material passing 200 Mesh	Type of soil: L. L.	Type of soil: P. I.	Type of soil: Classification	O.M.C. per cent	Proctor's density	C.B.R. test of soil only compacted at Proctor's density and immersed in water for 4 days: C.B.R. value per cent	C.B.R. test of soil only …: % water after 4 days immersion	C.B.R. tests on soil treated with lime compacted at Proctor's density and immersed in water for 4 days: % of lime	C.B.R. tests on soil treated with lime …: C. B. R. value per cent	C.B.R. tests on soil treated with lime …: % of water
Newly done embankment in subsection Alampur to Mile 3/5 of NH 6. (Dist. Howrah).	100	66.5	34.7	A_7 (Black cotton type)	22	83 lb/cu. ft. 1.33 G.M./C.C.	1.6	38	3	54	26
Krishnagar - Karimpur-Sikarpur Road. (Dist. Nadia) (39 M. 3 Furlong).	100	52	22	A_7 (Black cotton type)	18	90 lb/cu.ft. 1.44 gm./cc.	3.7	27	2.5	36.2	24
Do.									5	80	21

7.4. One of the essential points to be considered in the construction of stabilised soil base is the proper field control. This requires a mobile laboratory or field testing kit to be set up in tent which can be shifted with the progress of work. The essential tests required to be done in the field are (a) Sand content and P. I. of the soil and the soil-sand mix, (b) Proctor's test for determination of O. M. C. and maximum density of the mix, (c) Density and moisture content of the compacted layer after final rolling. The dry-density of the compacted base should not be less than 115 lb per cu. ft. and not below 95 per cent of the Proctor's density.

8. SOIL-CEMENT ROADS

8.1. Soil stabilisation with cement is a process in which naturally occurring soil or soil blended with a granular material is mixed with a little percentage of ordinary Portland cement and the mix compacted with the requisite amount of water to give a stable material known as soil-cement which hardens progressively as the cement hydrates. This improves and maintains the load bearing capacity and resists weathering of the natural soil even under adverse moisture conditions. The amount of cement required for soil-cement roads might vary from 5 to 10 per cent depending on conditions. Thus more than 90 per cent of the construction material is locally available soil and consequently the cost of construction of soil-cement roads is lower compared to other types of construction.

8.2. The method of construction of soil-cement roads is now fairly well known and as such will not be described here. However, a few salient points to be considered will be mentioned.

8.2.1. Generally speaking a soil suitable for stabilisation should have the percentage of material retained on 200 mesh not less than 50 per cent and the P.I. not more than 18. The plasticity index of a soil is, however, substantially lowered as an effect of the addition of cement[12] and for this reason soils having higher P.I. can also be stabilised with cement where use of higher percentage of cement is more economical than lowering the P.I. of soil by admixture of granular material. Soils containing high organic content and high sulphate content should be avoided.

8.2.2. The minimum amount of cement required for satisfactory stabilisation of a particular type of soil will depend on the type and physico-chemical properties of the soil. This amount is estimated by determining, in the laboratory, the compressive strength of soil-cement test specimens of standard size of 2 in. dia. and 4 in. long prepared by using different percentages of cement, by compacting to the predetermined density at the optimum moisture content and after damp curing for 7 days. The compressive strength for a mix suitable for the bottom layer should be above 250 lb per sq. in. and for the top layer it should be above 400 lb per sq. in. These suggested limits are, however, sometimes deviated from depending on various factors.

8.2.3. When it is proposed to lay a tar or bituminous chipping carpet over the soil cement layer, without any intervening layer of metalling, it is advisable to graft some amount of stone metal (about 7 to 12 cu. ft., depending on size, per 100 sq. ft. of surface) on the surface of the soil-cement during rolling. This will facilitate the adhesion of the carpet with the soil cement layer and prevent spreading or stripping of the carpet under effects of traffic.

8.2.4. In soil stabilisation with cement one difficulty was in dealing with heavy clays owing to the effort required to break down the clay and disperse the cement uniformly through it. Laboratory investigations[13] had shown that stronger and more water resistant material was obtained as the particle size of the clays was reduced. It was, therefore, desirable that the maximum size of clay particles should not exceed about 3/16 in.

8.2.5. A few typical examples of soil-cement roads constructed in West Bengal on experimental scale are shown in Appendix 11 (i to iv).

9. ROAD FAILURES—THEIR CAUSES AND REMEDIES

9.1. Due to the enormous increase in the intensity of traffic the occurrence of road failures had become more frequent in these days. Some roads which had stood well for years in the past had now shown signs of depression while some other newly constructed village roads, which were designed to carry light traffic, had already failed due to the unrestricted flow of heavy traffic which had suddenly grown up on the completion or improvement of such roads.

9.2. Of the many causes of road failures, all of which are not described in this Paper, one of the major causes was the failure of the subgrade with consequent depression of the road

surface. The sub-grade failure may be of the following categories:

(i) Due to natural settlement of a newly constructed embankment,

(ii) Due to yielding of the subgrade when the applied stress due to traffic increases beyond the safe bearing capacity of the subgrade,

(iii) Due to alternate swelling and shrinkage of highly expansive (e.g. black cotton) soil constituting the subgrade.

When the particular cause of the subgrade failure can be correctly ascertained, it is possible to suggest economic remedies for the repair and prevention of further failure of the road.

9.3. The Roads and Buildings Research Institute of West Bengal had occassions to study some typical cases of road failures in the State and it would be worthwhile to describe a few of these cases here.

9.3.1. **Belmuri—Bhandarhati Road** (Dist. Hooghly)

It was observed during the monsoon of 1957 that there occured bad settlements under traffic in the third mile of this road. The total thickness of the pavement was only about 10 inches laid over a clayey (A_7 type) subgrade. Undisturbed samples of the sub-grade soil were subjected to C.B.R. tests. The C.B.R. value was only 3.1 per cent based on which the requirement of pavement thickness was estimated to be about 18 in. for a traffic intensity of 45 to 150 vehicles exceeding 3 tons per day. It was recommended to lay about 8 in. thick hard crust over the existing damaged metalled surface.

9.3.2. **56th and 57th mile of Grand Trunk Road** (N.H.2)

Heavy subsidence of the road crust took place in these two milages of the road and this could not be prevented inspite of repeated repairs. The failure was due to low bearing capacity of the sub-grade soil and inadequate thickness of pavement for the heavy traffic on this road. Undisturbed samples of sub-grade soil were collected from the damaged (57th mile 3rd furlong) as well as from the undamaged (58th mile 3rd furlong) portion of the road. The test results are given in Table 6.

The C.B.R. tests showed that for the type of soil in the damaged portion, the requirement of pavement thickness was about 22 inches for traffic intensity of 450 to 1500 vehicles exceeding 3 tons per day, while that for the undamaged portion is

TABLE 6

Location	Condition of road	Existing thickness of hard crust	H.F.L.	Type of soil	C.B.R. value under saturation per cent	Moisture content at C.B.R. test per cent
57th mile 3rd furlong	Damaged	14 in. consisting of double brick flat soling 6 in. metalling 2 in. carpet.	In level with formation	$\frac{A_7}{CH}$	3.2	28
58th mile 3rd furlong	Undamaged	do	2ft 0 in. below formation level	$\frac{A_6}{CL}$ (Sand content =25 per cent	7.3	20

about 14 inches. It was suggested that in the damaged portions of the road the crust thickness be increased to at least 24 inches to make it adequate for increased traffic.

9.3.3. **Contai-Belda Road** (Dist. Midnapore)

Similar type of failure took place in this road also in several places within the 14th to 24th mile where the type of soil was clayey. Test results on two typical sub-garde soils from the damaged portions are shown in Table 7.

TABLE 7

Location	Existing thickness of pavement	Type of subgrade soil				C.B.R. tests under saturation for 4 days	
		L.L.	P. I.	Classification	Nat. moisture content per cent	C.B.R. value per cent	Per cent moisture at test
19th mile 8th furlong	About 12 in.	61	30	$\frac{A_7}{CH}$	34.5	2.2	41
24th mile 6th furlong	do	65.6	34.6	$\frac{A_7}{CH}$	25.7	4.3	26.4

The requirement of pavement thickness is at least 18 to 20 inches for a traffic intensity of 45 to 150 vehicles exceeding 3

tons per day and hence it was recommended to provide at least another 8 inches of hard crust.

9.3.4. **Badu-Kharibari Road** (Dist. 24-Parganas)

For a length of about 1½ miles the road crust was badly damaged during the monsoon of 1958. The earth work for embankment was done in 1955 and the road was completed in 1957. The total thickness of pavement was about 11 inches consisting of double brick flat soling, about 3 in. consolidated jhama khoa, about 2 in. consolidated stone metalling and surface painting. The traffic intensity under which the road failed was of the order of 50 to 70 heavily loaded buses and trucks per day. A sample from the subgrade in the worst affected portion was tested for C.B.R. value and other properties. The results are given in Table 8.

TABLE 8

Natural Moisture content	= 29.5per cent (In Dec. 1958)
Natural Dry density	= 83.7 lb/cft.
Liq. limit	= 52.5 per cent
Plasticity Index	= 26.1 per cent
C.B.R. value after saturation for 4 days.	= 4.0 per cent
Moisture content after 4 days saturation	= 31.2 per cent

From these results it was evaluated that the minimum thickness of pavement required on such soil for a traffic intensity of 45 to 150 vehicles (exceeding 3 tons) per day is about 15 inches.

9.3.5. **Barasat-Basirhat Road** (Dist. 24-Parganas)

Some portions in the sixth mile of this road were behaving badly under traffic and settlements of the pavement had taken place. The existing thickness of pavement was about 12 in. Undisturbed samples from the subgrade at 2nd furlong of 6th mile were tested and the results are given in Table 9.

TABLE 9

Nat. Moisture content	= 17.6 per cent
Liq. limit	= 58.5 per cent
P.I.	= 31
C.B R. value after 4 days saturation	= 3.9 per cent
Moisture content after 4 days saturation	= 23.7 per cent

The required thickness of pavement for a traffic intensity of 45 to 150 vehicles (exceeding 3 tons) per day was about 15 in. The pavement failed due to inadequate thickness.

9.3.6. **Saptagram-Tribeni-Kalna-Katwa Road** (Dist. Hooghly Burdwan)

In the 32nd and 33rd miles of this road there was uneven subsidence of the road embankment even under very low traffic. The earthwork for embankment was done in 1952 and the metalling was completed in 1954. The thickness of pavement was about 13 inches consisting of double brick flat soling and 7 inches compacted thickness of jhama khoa, stone metal and premix carpet. The height of embankment in the damaged portions was about 4 ft-6 in. to 5 ft whereas in the adjacent portion where no damage had taken place this height was about 1 ft-2 in. The subsidence of the road surface in this case was apparently due to settlement of the newly constructed embankment and not due to indequate thickness of pavement as practically no heavy traffic passed over the road before settlements occurred. The properties of the sub-grade soil in the damaged and undamaged portions of the road are given in Table 10.

TABLE 10

Location	Height of embankment	O.F.L.	H.F.L.	L.L.	P.I.	Nat. moisture content	Nat. Dry density	C.B.R. value per cent	M. Content at C.B.R. test per cent
I. 362 ft from culvert No. 32/1 (Undamaged portion).	1 ft 9 in.	1 ft 3 in. below formation.	1 ft 0 in. below formation.	45·6	18·6	15 per cent	98 lb/cu. ft. (1.57 gm./cc)	6.0	26
II. 1423 ft from culvert No. 32/1 (Damaged portion).	4 ft 6 in.	1 ft 6 in. below formation.	1 ft. 3 in. above existing road surface	50	22	17.5	95 lb/cu. ft. (1.52 gm./cc)	5.2	29.7

9.3.7. Other examples of pavement failure due to the settlement of newly constructed embankment, which had not been adequately compacted, have been described in para 4.1.3.

10. TRAINING OF PERSONNEL

10.1. During the construction of stabilised soil roads and also in other road problems calling for the application of soil mechanics, it was felt that the field supervisory staff requires some back ground knowledge of elementary Soil Mechanics and the principles of soil stabilisation for proper field control and correct application of the findings of the Research Institute in actual practice. In order to fulfil this requirement arrangements were made for the training of the Departmental Assistant Engineers and Overseers in the Roads and Buildings Research Institute. Though these short courses of training were mainly concentrated on the training of technical personnel in the principles and practice of soil stabilisation, the fundamental principles of Soil Mechanics as applied to roads were also included in the syllabus. Only 5 to 6 candidates were taken in each batch and the training for each batch lasted for six days and consisted of theoretical lectures for 3 hours and practical experiments for 3 hours daily. In addition to this, demonstrations of each operation of construction and field control tests during actual construction of stabilised soil roads, were given to as many of the candidates as conveniently possible. So far 17 Assistant Engineers and 36 Overseers of the Department had been trained. Recently, soil testing units with the most essential equipment had been started in some of the construction Divisions and the trained personnel were taking advantage of the equipment in applying their knowledge in practice.

11. CONCLUDING REMARKS

The foregoing examples and discussions show the importance of the application of Soil Mechanics in road construction and maintenance. Though the subject had been introduced recently in the field of Road Engineering in India, it had made very rapid progress elsewhere and the Road Engineer was finding it increasingly necessary to equip himself with the knowledge of Soil Mechanics and to arrange for the scientific investigation of his problems and the application of the results in actual practice. The durability of the road and the economic way of its construction and maintenance by the application of the simple principles of Soil Mechanics have been proved beyond any question by many workers in this field and all Roads Organisations in India and elsewhere have realised the necessity of establishing Institutions for the scientific study of the road problems by the application of these principles.

ACKNOWLEDGMENT

The Authors wish to express their thanks to Shri R.C.Roy, Chief Engineer, Development (Roads) Department, Government of West Bengal for his keen interest in the subject and kind

permission to publish the Paper and to Shri N.N. Majumdar, Deputy Director, Roads & Buildings Research Institute, Government of West Bengal, for his valuable advice and suggestions in connection with the investigations carried out. Thanks are also due to the Executive Engineers in charge of the constructions who offered full facilities and co-operation in conducting the field experiments and supplied valuable information particularly about the actual costs.

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Appendix 1

Revised U. S. Public Roads Administration (P.R.A.) System (1945) Classification

General Classification	Granular Materials (35% or less passing No. 200 sieve) A.S.T.M.							Silt-Clay Materials (more than 35% passing No. 200 sieve)			
	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-A	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis, percentage Passing:											
No. 10 sieve (ASTM)	50 max.										
No. 40 sieve (ASTM)	30 max.	50 max.	51 min.								
No. 200 sieve (ASTM)	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristic of fraction passing No. 40 sieve											
Liq. Limit (per cent)			N.P.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity Index (per cent)	6 max.			10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.*
Group Index	0		0	0		4 max.		8 max.	12 max.	16 max.	20 max.
Usual types of significant constituent materials	Stone fragments gravel and sand		Fine Sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as a Sub-grade	Excellent to good					Fair to Poor					

* Plasticity Index of A-7-5 subgroup is equal to or less than liquid limit minus 30.
Plasticity Index of A-7-6 subgroup is greater than liquid limit minus 30.

Appendix 10

Experiments on Soil-Sand-Shingle Stabilisation on the 5th mile of Basirhat—Jorai Road

(Dist. Cooch-Behar)

The sub-grade soil was of A 7-5 type and the sand locally available was of F.M.=0.9. Various graded mixtures of soil and sand were tested and a mix of soil-sand=1 : 2 gave the best results (P.I.=7.0 ; Proctor's density =106 lb per cu. ft.)

An experimental stretch of 100 ft of base course was laid in 1953 using an admixture of equal parts (by vol.) of shingles (3 in. down) with soil-sand mix and compacted to 5 in. thickness. This was overlaid with 6 in. shingle consolidation. The cost of the stabilised base course came to Rs 15,200 per mile which is much lower than even single brick soling (Rs 23,760 per mile).

Experiments on Soil-Sand-Jhama khoa stabilisation on Dinhata—Gossaimari Road

(Dist. Cooch-Behar)

The sub-grade soil on this road was of A5 type and the sand available was of F.M.=0.9. Two stretches of 1 furlong each were laid with the specifications of (i) 1 part soil : 2 parts jhama Khoa (1 to 2 in. size) compacted to a thickness of 5 in. in the portion 3rd mile 8th furlong and (ii) 1 part soil : 2 parts sand : 4 parts jhama khoa compacted to 6 in. thickness in the 3rd mile 5th furlong. The cost worked out as Rs 31 per 100 sq.ft. for specification (i) and Rs 32 per 100 sq.ft. for specification (ii) (i.e. Rs 19,642 and Rs 20,275 per mile respectively for a 12 ft width of road). The work was done in June 1953 and was left without any covering during monsoon. There was heavy bullock cart traffic upto 700 Nos. per day under which the road failed.

Appendix 11

Experimental Soil-Cement Roads in West Bengal

(1) Jhargram — Tangra Road (Dist. Midnapore)

Soil-cement construction on this road was done in 1953. The sub-grade soil was of A4 type (sand content=52 per cent P.I.=5) and was considered to be suitable for stabilisation with cement. 6 per cent (by weight) cement was used and the soil-cement layer was compacted to 120 lb/cu. ft. dry-density at an optimum moisture content of 12 per cent. The soil cement layer was covered with $1\frac{1}{2}$ in. premix carpet. The experimental portion of the road has been behaving fairly satisfactorily except that the grip between the premix carpet and the soil-cement layer was not satisfactory and the carpet under the two tracks appeared to have moved laterally thinning out the wearing course and some patches came off in several places.

(2) Contai—Daruamaidan Road, Digha Foreshore Road and Digha Beach Approach Road (Dist. Midnapore)

These roads are near the seashore and the natural sub-grade is pure sand of F.M.=1.26. It was necessary in these cases to mix a clayey soil which was available within a mile. A mix proportion of sand : soil = 2 : 1 (P.I. = 6 ; O.M.C. = 15 per cent and Proctor's density = 115 lb cu. ft.) was found to be suitable. The specifications adopted and costs were as follows:

Layer	Compacted thickness	Percent of cement used	Cost per 100 sq. ft. Rs
Bottom layer	6 in.	5%	24.25
Top layer	4 in.	8%	24.06
Premix carpet (Over a seal coat)	$1\frac{1}{2}$ in.	—	40.00

Total cost = Rs 88.31 per 100 sq. ft.

i.e. Rs. 46,628 per mile for 10 ft. width

The cost of construction, in this region, with the conventional specifications of double brick flat soling, brick-on-end edging and $4\frac{1}{2}$ in. (loose) jhama consolidation, 3 in. (loose) stone consolidation and 1 in. premix carpeting works out to about Rs 131.00 per 100 sq. ft. (i.e. Rs 69,168 per mile for 10 ft. width). These roads were constructed in 1956-57 and have so far stood well.

(3) Bolepur—Suri Road (Dist. Burdwan)

Moorum is available in plenty in this region and a few field experiments on the stabilisation of moorum with cement were carried out in

March-April, 1955 on the 2nd mile of this road. The general type of the moorum was as follows:

Mechanical Analysis						P. I.	O.M.C.	Proctor's dry-density
Percent retained on A.S.T.M. sieve Nos.								
7	10	30	44	100	200			
57.6	63.4	69.3	71.0	73.4	75.00	12	11 per cent	125 lb per cu. ft.

Laboratory tests were conducted to find out the compressive strength of moorum cement blocks stabilised with different percentages of cement and damp-cured for seven days with the following results:

Per cent of Cement —	5	6	7	8	9	10
Compressive strength lb per sq. in.	280	373	410	437	560	653

The following experimental section were constructed.

Section	Specifications	Cost per mile Rs
1.	6 in. compacted moorum cement with 10 per cent cement.	19,008
2.	6 in. compacted moorum cement with $8\frac{1}{2}$ per cent cement.	16,808
3.	9 in. compacted (in 2 layers) of moorum cement, bottom 5 in. with 5 per cent cement top 4 in. with 10 per cent cement	23,016
4.	6 in. boulder soling with 3 inch moorum topping.	19,437
5.	6 in. boulder soling and $4\frac{1}{2}$ in. (loose) stone metal consolidation.	35,538

Sections 1 to 4 were covered with $1\frac{1}{2}$ in. thick bitumen carpet at the cost of Rs 14,600 per mile and section 5 was covered with surface painting at the cost of Rs 7,300 per mile. The experimental portions of the road are behaving well under heavy traffic.

(4) Barasat—Basirhat Road (Dist. 24 Parganas)

Increase in traffic necessiated the widening of this road from the original 8 ft width to 16 ft width by constructing 8 ft width of road on the left flank. A portion of 3 furlongs in the 11th mile of this new construction was selected for experimental soil-sand and soil-sand-cement construction. The specifiacations adopted and costs were as follows :

Section	Bottom layer	Top layer	Surfacing	Cost per mile Rs
I. 6th furlong	6 in. compacted thickness soil : sand=1 : ½	3 in compacted thickness Soil : Sand =1:1 with 1½ in. to 2 in. stone metal grafting @ 7 cu. ft./ 100 sq. ft.	1½ in. pre-mix carpet	58,336
II. 7th furlong	6 in compacted thickness soil : sand=1 : ½ with 6 per cent cement	3 in. compacted thickness Soil : Sand =1 : ½ with 7 per cent cement stone metal grafting as above	do.	70,136
III. 8th furlong	Double brick flat soling	4½ in. (compacted) W.B. Macadam	do.	75,958

The construction was done in March-April 1957. The traffic is fairly heavy. Sections II and III have been behaving well but in section I there was some damage to the road which was repaired in October, 1958.

(5) **Milan Sangha Road** (Dist. 24 Parganas)

The entire length of 1 mile 1.54 furlong of this road was constructed with soil-cement stabilisation during 1956–57. Mainly two types of specifications (with minor alterations) were adopted viz.

(a) 6 in. (compacted) soil-sand-cement using 6 to 8 per cent cement covered with 2 in. premix carpetting.

(b) 6 in. (compacted) soil-sand-cement using 6 to 8 per cent cement covered with 3 in. (compacted) W. B. macadam with jhama khoa and 2 in. premix carpeting.

The average cost per mile for specification (a) was Rs 30,927 and that for the specification (b) was Rs 41,416. The traffic intensity is light consisting mainly of bullock carts, cycle rickshaws and private cars. The road has been behaving well.

Paper No. 214

"DESIGN AND CONSTRUCTION OF TRAINING WORKS OVER RIVER GHOGRA AT AYODHYA"

By

N. C. BHARGAVA*, B.SC., C.E. (HONS), D.I.C.

CONTENTS

SYNOPSIS

The Paper describes the construction of River Training Works over river Ghogra at Ayodhya. These bunds are part of the scheme of bridging this river in mile 84 of Lucknow-Gorakhpur Road, also known as O. T. Road, National Highway No. 28. The bridge will have a waterway of about 3,110 feet. While the guide bunds had been completed this year, the construction of the main bridge is scheduled to be taken in hand very shortly.

Details of design, construction, difficulties met with and organisational problems have been dealt with in this Paper.

1. INTRODUCTION

The National Highway No. 28 which crosses river Ghogra at Ayodhya in Faizabad District in the 84th mile from Lucknow has at present no permanent bridge. As a matter of fact, there is no road bridge yet over the river Ghogra which flows through the heart of Uttar Pradesh dividing the eastern and the western parts of the State. Every year a pontoon bridge designed for a

* Engineering Construction Corporation, Bombay-1 [formerly Executive Engineer, Temporary Bridge Construction Division, P.W.D., Faizabad (U.P.].

maximum load of 5 tons permitting one way traffic is erected on this crossing during the fair weather. The Khadir of river Ghogra, popularly known as Saryu, is very wide. Every year after rains, while most of the discharge flows through the main channel, there are at least two other subsidiary streams which also have to be bridged. On an average about 2,400 feet length of pontoon bridges is erected every year. These different pontoon bridges are connected by tracks a mile or so in length. Due to the narrow roadway on these pontoon bridges and the one lane tracks, traffic hold-ups at this crossing are very frequent and are a usual feature. Even in the best weather, it takes about an hour and a half to cross this river.

The position becomes worse during fairs when thousands of people from all parts of India come to Ayodhya, a place of religious importance. During this period all vehicular traffic is stopped over the pontoon bridges when it is needed most.

During rains when the pontoon bridge is dismantled and the otherwise serene waters of the Saryu swell up, steamer and barge service is put in operation. But on certain occasions due to bad conditions of weather or river necessitating frequent changes in the siting of jetties, no barge or steamer is able to ply, causing great deal of hardship to the growing traffic. The necessity for a high level bridge at this crossing is, therefore, very great.

2. SITE

In spite of the importance of Ayodhya, no railway crossing yet exists over the Saryu at Ayodhya. While the right bank of the Saryu at Ayodhya is fed by the Northern Railway, the other bank of the river has the terminus of the N.E. Railway. It had at first been contemplated to provide a road-cum-rail bridge at this crossing, and with this view several bridge sites upstream and downstream of the town of Ayodhya were investigated and surveyed. But as the Railways were not in a position to agree to the proposal, it was decided to go ahead with the road bridge near the site of the present pontoon bridge where the river is known to flow since ages and where the length of the training works would be minimum.

3. DISCHARGE AND WATERWAY

3.1. **Discharge**—The Ayodhya bridge and the guide bunds have been designed for a discharge of 10,00,000 cusecs which was worked out by taking five cross sections of the river upstream and downstream of the site. The discharge was also computed from the catchment area of 33,500 sq. miles. Computations were also made for the flood discharge of three railway bridges, Elgin, Turtipur and the Inchcape.

3.2. **Waterway**—These three railway bridges have a waterway of 3,400, 3,000 and 3,600 ft respectively, the last two bridges are downstream of Ayodhya.

The Khadir of river Ghogra is about 2 miles wide, but at the site selected the active stream runs about 4,000 ft wide.

The bed of the river consists of sand which is easily erodible. At first a clear waterway of 3,250 feet was decided and the design of the guide bunds had been worked out accordingly. Subsequently with the slight change in alignment, as discussed in para 4, the distance between the centres of the two abutment wells was kept as 3,213 ft, giving a waterway of about 3,100 feet.

4. ALIGNMENT OF GUIDE BUNDS

Based on the survey done in 1955, the alignment of the bunds was kept almost parallel to the river flow and the bridge kept at right angles. Whereas the Lakarmandi side bund had straight shank and regular curved noses on the upstream and downstream end, the shank of the Ayodhya side bund after being straight for 1,100 feet was joined with the existing banks and ran along them for certain length. This, of course, caused a little fluming at the upstream end, which was not considered objectionable in view of the recommendations of Mr. Bell, substantiated by experiments carried out by Mr. Haughton where a little constriction at the upstream end was found to straighten the current and give more uniform scour at the bridge site.

Change in the above alignment, just before the construction started, became necessary as it was discovered that the direction of flow had shifted more towards the eastern side than before and that the water depth on the proposed alignment of Ayodhya side had risen to as much as 10-15 feet. To avoid laying the apron in deep water, the straight shank and downstream curved nose of the Ayodhya side bund was swerved round about point C, towards the town of Ayodhya, Plate I. By doing so, the water depth had been reduced in a major portion to 5 feet. The Ayodhya side bund was ultimately built on this alignment.

The above change in the alignment of the Ayodhya side bund also necessitated certain changes in the alignment of the bund on the opposite bank. Keeping the straight shanks of the two bunds parallel and the axis of the bridge at right angles, it was observed that the upstream curved nose of the bund fell in a region which was pretty high and dry, with very little chance of the river sticking to this bank in the coming years. It was consequently proposed to align the Lakarmandi bund at 86° to the axis of the bridge bringing the upstream nose towards the river by 157 feet. This alignment stood at the end of 1958.

Before the work on the bund was, started, it was felt that the decision to keep the bunds inclined at 86° to the bridge axis was not very judicious and that this could be avoided by keeping the upstream nose more or less the same as in 1958 and reducing the waterway to some extent. The alignment ultimately decided and adopted is shown by firm lines in Plate I. As stated earlier, the width required for the design discharge was less than the waterway of 3,250 feet initially kept. Larger waterway was kept with a view to bridge the active stream, but now that full scale river training works were being built, slight curtailment in the waterway necessitated due to change in alignment was considered immaterial.

5. DESIGN

5.1. **Length of the Bund**—The waterway for the bridge having been decided initially as 3,250 feet, the lengths of the guide bunds were kept as below:

Length upstream 1.2 L $= 1.2 \times 3250 = 3900$ ft

Length downstream 0.2 L $= 0.2 \times 3250 = 650$ say 700 ft

The above co-efficients were adopted after weighing carefully the recommendations* of Mr. Spring, Mr. Bell and studying the behaviour of the guide bunds built at the railway bridge and numerous model studies done at the Power Research Station, Poona, and the Irrigation Research Institute, U. P. The original drawings were prepared accordingly. On further consideration the upstream length of the guide bund on the Lakarmandi side was kept equal to the originally, decided waterway i.e., 3250 feet. The length actually constructed is 3,247 ft 9 in.

5.2.—**Section of the Guide Bund** Keeping in line with the normal practice the width of the bund was kept as 20 feet for facility of transport of materials.

1. * River Training and Control—F. J. E. Spring (Govt. of India Technical Paper No. 153).

2. The Continuous Bund and Apron Method of Protecting the Flanks of Bridges in the Punjab—J. R. Bell (Govt. of India Technical Paper No. 2B of 1890).

Originally it was proposed to provide a side slope of 2:1 on either side of the straight portion of the bund and to provide a slope of 3:1 on the curved portions, but since the slopes were being pitched adequately, a uniform slope of 2:1 was provided throughout.

5.3. **Pitching and its Thickness**—For sandy beds with a river slope of 9 inches per mile, thickness of pitching recommended by Mr. Spring is 37 inches. Pitching 3 feet thick had consequently been adopted. The three feet thickness consisted of 3 inches thick fairly compacted brick-ballast, a coat of flat bricks and 2½ feet thick stone boulder pitching. Brick-ballast and bricks were provided underneath in order that the bund may act as an inverted filter so that the core of the bund which was built with sand due to non-availability of clay in the vicinity might not be washed with wave action.

5.4. **Launching Apron**—The apron was designed to withstand a scour equal to 2 times the normal scour at the upstream curved nose and 1.5 times the normal scour for the straight and down-stream nose.

With $Q = 10,00,000$ cusecs

$f = 0.9$ (as determined by Research Station, Roorkee).

$$\text{Normal scour } D = 0.473 \left(\frac{Q}{f}\right)^{\frac{1}{3}}$$

$$= 0.473 \left(\frac{10,00,000}{0.9}\right)^{\frac{1}{3}}$$

$$= 49 \text{ feet.}$$

$2D = 98$ feet and $1.5D = 74$ feet.

The width of the apron on the other hand was kept 1.5 times the greatest depth of scour as measured from low water stage,

H.F.L. being R.L. 310.00

L.W.L. being R.L. 294.00

For the nose depth of scour L.W.L. $= 98 - (310 - 294) = 82$ feet and for the straight portion $74 - (310 - 294) = 58$ feet.

The widths were consequently kept as $1.5 \times 82 = 123$ feet, say, 120 feet and $1.5 \times 58 = 87$ feet, say, 90 feet.

Thickness of Launching Apron

In order that, when scoured, the apron may launch itself at a slope of 2 : 1, the thickness required for the entire apron works out to 1.5 times the thickness of pitching. To allow for irregular launching, it is normal to increase the quantity of boulders in the apron by 25 per cent, in such a way that the thickness of the apron may increase uniformly from the inner edge to the outer edge of the entire width of the apron. Based on these considerations, the thickness of the apron at the inner edge was kept as $1.5 \times 3 = 4.5$ feet, and at the outer edge $2.25 \times 3 = 6.75$ feet.

Since, however, the waterway being kept for the bridge was more than the regime width required for the discharge of 10,00,000 cusecs and maximum scour was being catered for, it was decided that while the width of the apron may be kept the same as worked out on the theoretical considerations, the excess quantity of boulders required for irregular launching should be dispensed with in the initial stages and instead of a uniform thickness, the apron may be kept 3 feet thick at the inner edge and 6 feet thick at the outer, giving an average of 1.5 times the thickness of pitching as theoretically required.

5.5. **Curves**—For a gentle slope of as little as 9 inches per mile and a scour exceeding 50 feet, curves of 800 ft radius were provided at the upstream and downstream noses. The angles kept were 135° at the upstream nose and 45° at the downstream.

5.6. **Ayodhya Side Bund**—While the design of the downstream portion of the Ayodhya side bund was the same as on the other side, the length of the bund on the upstream side had been greatly reduced due to the presence of existing high banks as shown in the index plan, Plate II. Under normal circumstances, it should not have been necessary to extend the bunds upstream of Laxman Ghats, as the banks are very high, but with both the bunds thrown in the river bed, there were always chances that heavy scour might take place in the area adjoining these banks and thus cause serious damage to the numerous buildings and temples existing there. To safegaurd these banks and also to provide bathing facilities for the pilgrims, the guide bund had been extended from Laxman Ghat to Gola Ghat. A length of 1300 feet of the bund had, however, been built in the shape of ghats with steps and landings leading to the river bed. It was proposed to concrete and stone slab these steps and landings after a couple of monsoons when the slopes and the apron laid had been stabilised. These new ghats had become absolutely necessary, since with the construction of the bund downstream of Laxman Ghat, the old existing ghats extending upto Naya Ghat had been left high and dry.

6. SPECIFICATIONS

6.1. **Earthwork in Embankment**—Due to non-availability of clay, the earthwork for the bund was mostly done in sand. The little quantity of clay available was used for overtopping the sand embankment so as to reduce the sand blowing on windy days.

6.2. **Earthwork in Water**—As the main stream of the Ghogra hugs its right bank, earthwork in quite an appreciable portion on the Ayodhya side bund had to be done in water depth ranging upto 15 feet.

To deflect the water current and reduce its force, the first step taken was to construct a couple of balli spurs 125 feet long and inclined at 60° to the up stream flow. For this purpse, 6 in. diameter ballies in two rows five feet apart were sunk upto a depth of 8 feet, the longitudinal spacing between two adjacent ballies having been kept as 3 feet. The space between the two rows was filled by brushwood, jhau poolas and gunny bags filled with sand. These spurs were quite effective in deflecting the current but could not stop the earthwork from being washed away.

The most effective remedy for stopping washing away of the earth was the construction of boulder walls at the toe of the embankment and carried upto the top of the water level. These toe walls subsequently formed part of the apron. For water depths upto 6-7 feet, the boulders for the toe walls were just dumped in as in the case of apron laying but for greater depths, these walls were built by filling cages with boulders. After the earthwork was done, the additional boulders laid in apron were salvaged and used in pitching work.

Photographs 7 and 8 show the construction of the embankment at Laxman Ghat where there had been a huge scour zone necessitating the construction of toe walls.

6.3. **Quality and Size of Boulders Used**—All the apron and the top $2\frac{1}{2}$ feet thick pitching was done with stone boulders brought from Jarwa and Katarnia Ghat on North-Eastern Railway and Shivalik ranges on Central and Northern Railways.

All stone used was sound limestone or quartzite such as will not weather on surface nor deteriorate after use. Stone containing soapstone, shale, unsound sandstone or any other easily disintegrable substance was rejected. Pieces used had sharply fractured edges and were free from admixture of earth and rubble.

The size of boulders was such that for the back slopes where 1 ft thick stone pitching was done, their lengths were about equal to the thickness of pitching with the backing so that the pitching could be done in one layer. On the front slopes of the bund where the thickness of pitching was $2\frac{1}{2}$ feet, the stones were laid as in masonry work and were well bonded. The stones were roughly hammered to ensure fitting of one on the other. No pitching or apron stone was less than six inches on any side and it was roughly ensured that no boulder piece weighed less than 65 lb and more than 120 lb.

6.4. **Laying of Pitching Stone**—The laying of stones was commenced from the toe of the embankment and proceeded upwards, each stone being laid by hand perpendicular to the slope, firmly bedded against the slope and against the adjoining stone, with the ends in contact and with well broken joints. The spaces between the larger stones and the slopes were filled with smaller pieces, reducing voids to minimum possible. For the back slopes where no apron was laid in front, small toe walls $1\frac{1}{2}$ to 2 feet deep were made to safeguard against slipping of stones.

6.5. **Laying of Apron in Dry Area**—Most of the apron specially in the Lakarmandi side guide bund was laid in an area which had gone dry after the rains. The apron stone in this region was simply dumped as it was received from the trucks, no attempts having been made to hand pack it. Before laying was, however, started the bed was levelled and bamboo poles 25 feet apart longitudinally and transversely, with the required height of the apron marked with paint, were embedded in position. The heights of the apron laid were checked by stretching 'moonj' strings linearly and diagonally and keeping the top of boulder flush.

6.6. **Laying of Apron in Water**—Most of the apron was laid in 2 feet to 6 feet deep water. All such portions were divided in grids of 25 feet × 25 feet or so depending on the width of water and dry land. The grids were demarcated at the site by fixing bamboos and ballies and running 'moonj' strings across them. Where depth of apron exceeded the depth of water, marks denoting the required height of apron were painted on the bamboo posts and boulders laid upto that level.

In cases where the depth of water exceeded that required for the apron but not more than 8 feet, walkways made of boats underneath and bamboo meshes on top were used for bringing boulders from the stacks to the respective grids. To facilitate measurements, boulders required for each grid were stacked separately and measured before laying. Strict supervision was exercised to ensure that the boulders measured were laid in the specific grids for which they had been collected. For avoiding any large pockets in the apron, the dumping of boulders in a particular grid was stopped several times in between and trained 'mallahas' made to dive to adjust the laid apron. The surface was also occasionally checked by means of boning rods.

Apron more than 8 feet deep

Boulders for apron laid in more than 8 feet deep water were laid in cages. Two types of cages, one of wire and the other of ballie frames, were used, Fig. 1.

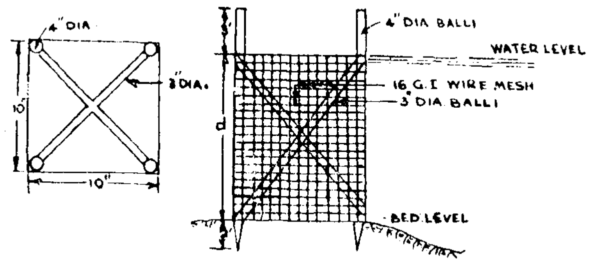


Fig. 1

Most of the wire crates were 10 feet × 10 feet with stout ballies at the corner suitably stiffened with diagonal braces. At a few places smaller units of 5 feet × 5 feet were also used. The corner ballies had tapered ends 2 feet lower than the bottom of the crate to ensure the embedment of the empty crates. The top ends of these ballies were left 3 feet to 4 feet higher than the water level for guiding the laying of these crates in position. The cage in itself was formed by 16 gauge G.I. wires interwoven in a mesh of 9 inches. No strands were fixed at the bottom which was left open. It was, however, discovered that these wire cages besides being too costly took a lot of time in fabrication which due to paucity of time could be ill-afforded. The wire cages were consequently replaced by ballie crates which were more or less of the same size and dimensions as the wire cages except that no vertical strands were provided as in case of the wire cages and that the horizontal strands were made of bamboos 2 in. to 3 in. diameter instead of 16 gauge wire. This change expedited the progress of the work considerably.

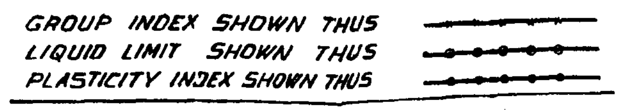
The launching of these cages was done by means of 2 boats tied together with ballies such that a gap of 12 feet or so existed between the two boats. In between this gap, the cage was placed on ballies which were removed when the boat was brought to the correct position. The dumping of the crates was followed by shaking them from their protruding ends so as to secure a firm grip in the bed of the river. This was followed by dumping the required quantity of boulders. The cage was held in position by ropes till about 100 cubic feet of boulders had been dumped in the crate. Care was taken to ensure that uniform thickness of boulders had been achieved in each cage. This was checked by putting bamboos fixed with flat piece of planks at its bottom

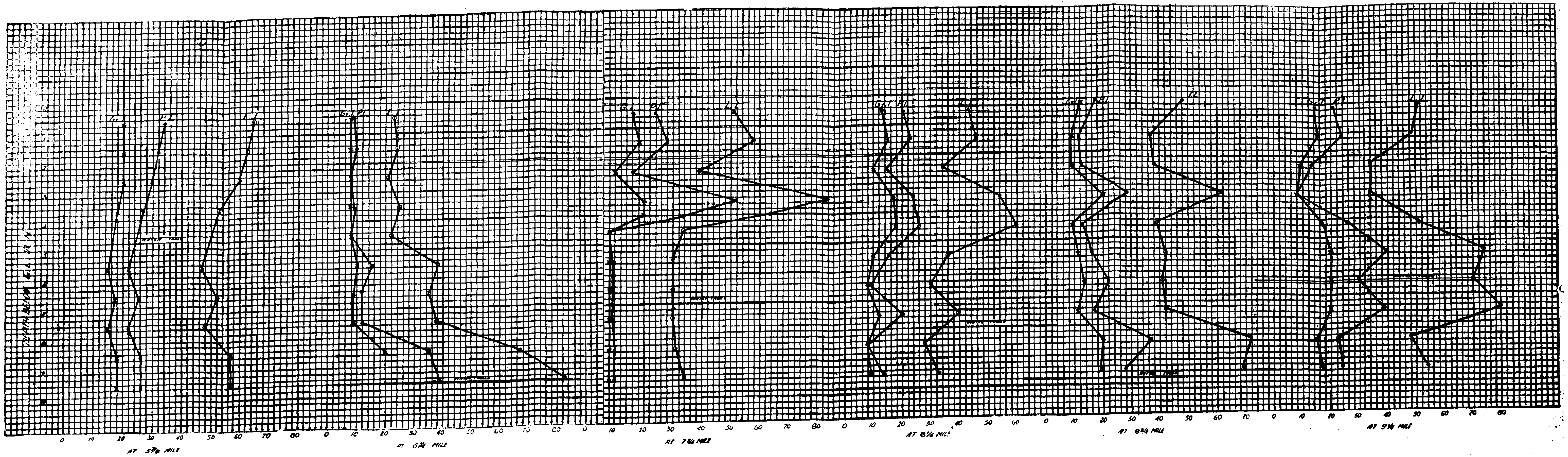
PLATE I

GRAPH SHOWING TYPE OF SOIL AT DIFFERENT DEPTH & MILEAGE OF N. H. 2

SECTION UTTARPARA KALIPUR ROAD TO NABAGRAM

DIST. HOOGLY





and checking the depth of water over the boulder layer as compared to the original depth.

6.7. **Kankar Pitching**—To save in cost, the three feet thickness of the pitching done on the 20 feet wide top of the bund consisted of 3 in. flat brick soling, $1\frac{3}{4}$ ft block kankar pitching and 1 ft stone pitching.

The kankar used was of hard silica variety, free from earth and showing bluish surface when broken. Like stone work, it was roughly dressed with hammer before it was laid. Regular thickness was ensured by means of profiles.

6.8. **Brick Ballast and Brick Soling**—To enable the bund to act as an inverted filter, a layer of $4\frac{1}{2}$ in. brick ballast consolidated to 3 in. was laid on the front earthen slopes of the bund. Flat bricks closely knitted were then laid over the consolidated surface of brick ballast. The bricks used were either straight overburnt or first class. No jhamas or under-burnt pieces were permitted.

7. CONSTRUCTION OF AYODHYA SIDE GUIDE BUND

It had been planned at first to construct the complete training work in one season, but as favourable response could not be had from firms which had sufficient resources and organisational capabilities to handle a job of such magnitude, it was decided to take up the work on Ayodhya side guide bund in one season and of the bund on the opposite bank in the other. Work on Ayodhya side guide bund was started in January 1958.

The quantities of principal materials used in construction of the Ayodhya side bund were :

(1)	Stone boulders	19.00 lakh cu. ft.
(2)	Block kankar	0.80 lakh cu. ft.
(3)	Bricks	10.00 lakh nos.
(4)	Earthwork	29.00 lakh cu. ft.

The total quantity of stone boulders required for the complete training works was 65 lakh cu. ft. requiring 10,000 broad gauge wagons. Heavy traffic like this could not be handled from the existing railway sidings at Ayodhya where the capacity was limited to 6 wagons. A private siding with a capacity to hold 120 wagons at a time was consequently built near the Ayodhya Station. The cost of this siding as likely to be paid to the Railways is Rs 92,000.

The actual construction of the Ayodhya side bund was started as late as February 1958—a great deal of time having been lost in setting up the organisation and making preliminary arrangements.

Due to the great difficulty experienced in carrying out the earthwork in water, considerable time was lost and, therefore, it was necessary to work round the clock. Subsidised canteens were opened at site to tempt the labourers to stay there. The laying of apron in deep water was another problem dealt with successfully as explained in para 6.6.

As explained in para 5.6, of the 2,635 feet length of the guide bund, 1,305 feet of the bund had been built in the shape of ghats with steps and wide landings. A typical cross section of these ghats is shown in Plate III. At first only the stone pitched steps were built and it was intended to concrete and stone slab these steps next year. Work on concreting the steps was started in 1959 but later it was decided to defer it till the bund was stabilised.

Proposals to extend the steps, build canopies, changing rooms, bath rooms, proper access to the ghats and an inspection house on these ghats were now being scrutinized.

8. CONSTRUCTION OF LAKARMANDI SIDE GUIDE BUND

8.1. Work on the construction of Lakarmandi side bund was commenced in January 1959, details in Plate IV. The bund on this side being 4,784 feet long was nearly double that on Ayodhya side and needed the following quantities of materials :

(1)	Earthwork (including that of Lakarmandi side approach road)	100 lakh cu. ft.
(2)	Stone boulders	46 lakh cu. ft.
(3)	Block kankar	1.5 lakh cu. ft.
(4)	Bricks	27 lakhs.

The contracts were finalised in late December 1958, and work orders were given in early January 1959.

Situated far away from Ayodhya in the heart of the river bed, constructional difficulties for the Lakarmandi side bund were different from the ones faced for the Ayodhya side bund.

The greatest problem to be tackled was the transport of boulders from Ayodhya across the river to the site of bund. The pontoon bridge built every year was very congested. Even in the dullest season, it had a traffic of about 2000 tons and with its load limitation of $4\frac{1}{2}$ tons, it certainly could not cope with the boulder traffic which during the peak period was as high as 7500 tons per day. Transport of boulders by boats was equally impossible. It was, therefore, decided to construct temporary pile bridges across the main stream and across the subsidiary streams met with.

8.2. **Pile Bridges**—Three pile bridges were constructed, one 1070 feet long across the main channel, the second 70 feet long across the subsidiary channel flowing through the heart of the proposed bund and the third 60 feet long across the Tehri Nala on the service road from Lakarmandi Station to the site of the bund. The first two pile bridges catered for the boulder traffic from Ayodhya on the Northern Railway, while the third for the traffic from Katra and Lakarmandi Stations on the North-Eastern Railway.

Capacity of Pile Bridges—All the three pile bridges were initially designed for a vehicle weighing $8\frac{1}{2}$ tons. When the boulder traffic was resumed, it was observed that most of the transport trucks available were diesel operated with unladen weight of 5 tons and it was not economical to operate them unless a laden weight of 10 to 11 tons was permitted on the pile bridges. Fortunately while designing the pile bridges, allowance for impact which is normally ignored for timber structures was taken into consideration. This error helped and vehicles with laden weights of 10 to 12 tons were permitted on the bridges with speed restriction of 5 miles per hour.

Carriageway—The carriageway on all the pile bridges was 10 feet wide. Obviously this was inadequate for two way traffic specially on the main pile bridge across the river. One way traffic was consequently permitted on this bridge in the initial stages. While all the laden trucks were passed over the pile bridges, the empties were passed over the pontoon bridge. It was, however, soon observed that this system was not working satisfactorily. The pontoon bridge itself being narrow and traffic sufficiently heavy, hold ups were too frequent and long, and the average of daily trips per truck was gradually falling. Two way traffic was subsequently introduced and barriers erected at each end of the pile bridge. This along with light arrangement during the night hours proved so satisfactory that there was no accident at the pile bridge and no truck had to be pulled back.

Bridge Details—The average span in each pile bridge was 10 feet, each span consisting of a 3 inch thick deck of planks

Photo 1.
15 H.P. diesel winch mounted on trolly for extraction of piles

Photo 2.
5 H.P. diesel winch mounted on boats for extraction of piles

Photo 3. **View of pile bridge with vehicle passing over the floating span**

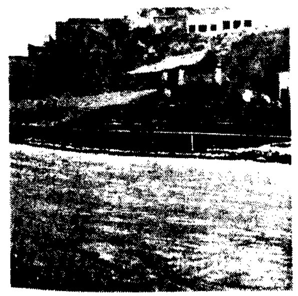


Photo 4.
View of a portion of completed ghat on Ayodhya side

Photo 5.
View of earthwork in progress on Ayodhya side in running water. The toe protected by boulder apron is visible

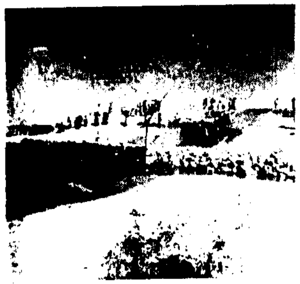


Photo 6.
Another view of earthwork carried out in water

Photo 7.
View of earthen approach road, Lakarmandi side being protected by boulder pitching and crates

Photo 8. **View of Lakarmandi side guide bund showing boulder pitching in progress**

Photo 9.
Another view of boulder apron partially submerged under water. In front is the subsidiary channel successfully blocked

Photo 10.
View of completed pile bridge

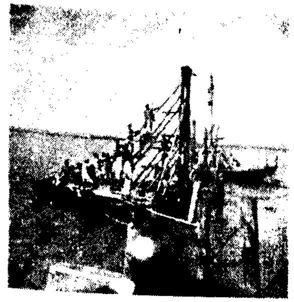


Photo 11.
View of manually operated pile driving equipment

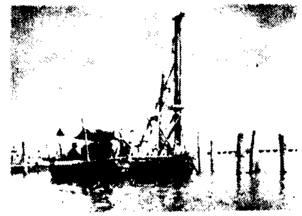


Photo 12.
View of mechanically operated pile driving set

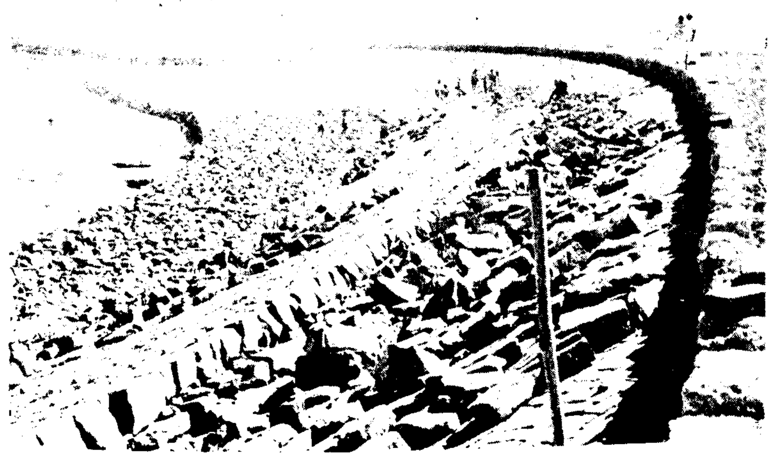


Photo 13.
Rear view of the downstream nose of Lakarmandi side guide bund

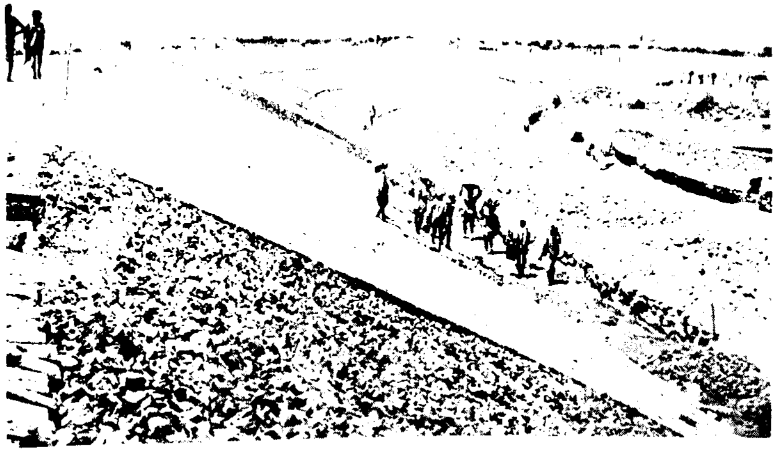


Photo 14.

View of Lakarmandi side guide bund showing earthwork, brick ballast and brick soling

Photo 15.

Boulder apron laid on Lakarmandi side guide bund

supported on five $7\frac{1}{2}$ inches $\times$ 10 inches longitudinal beams. The substructure for each span comprised three 8 inches to 10 inches diameter piles 25 feet to 35 feet long, tied on top with a cross beam 6 inches $\times$ $9\frac{1}{2}$ inches in cross section. Transverse bracings and struts as shown in Plate V were provided for rigidity. As an additional precaution stay piles at every 30 feet were provided on both sides in the portion where the channel was too deep. These stay piles were connected with the pile bridge by the help of bracings. For fixing the bracings under water, one end of the bracings was fixed with the piles before it was driven while the other end was fixed after all the three piles had been driven. The typical cross section of the bridge shows joint details and other salient features Plate, V.

Floating Span—The Ghogra being a navigable river, arrangement for one floating span capable of removal had to be made in the main pile bridge. The floating span had to be removed every Friday to pass the country boats plying in this stream. The details of the floating span are shown in Plate VI and are self explanatory.

A country barge of 14 tons capacity available at the ferry was utilised for supporting the floating span 45 feet long. This span was made independent of the remaining length of the bridge by driving additional row of piles adjacent to the pile supports of the end spans. The floating span consisted of five 8 inches $\times$ 6 inches R. S. joists supported on one end on the pile group and the other on the barge. To allow for the up and down movement of the barge, the girders were hinged with the cross beam on top of the piles with a $1\frac{1}{2}$ inch diameter pin, round which the girders could move. On the barge itself space for the horizontal movement of the girders was provided by making a rectangular slot 12 inches long in the 6 inches $\times$ 4 inches $\times$ $\frac{3}{8}$ inches angles supporting the pin which held the girders in position. For easy movement of the girder a steel plate was placed in between the angles. Decking on top and in between the girders was provided in the usual manner.

When the floating span was required to be removed, the decking planks and the cotter pins on both sides of the girder were taken out, the protruding girders slided on the boat and the boat moved to one side. After passing the navigation boats, the floating span was fixed in position by adopting the reverse course.

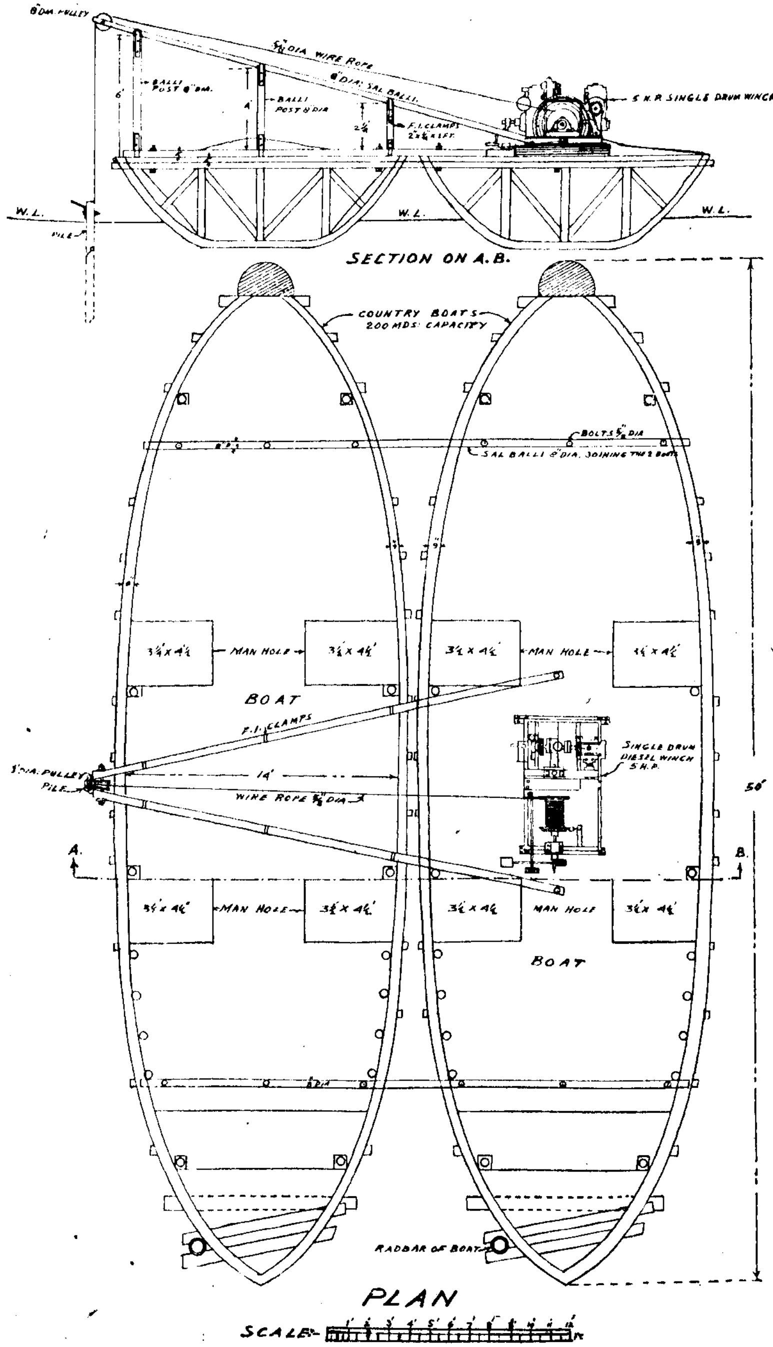
Pile Driving—Most of the piles were driven 18 to 20 feet below the river bed. Five derricks were set up. Of the lot, one was used exclusively for erecting the piles in position and the remaining four used for sinking. Except for one set which was fitted with a diesel winch of 5 H. P., all the remaining three derricks were operated by manual labour. The mechanically operated derrick was used mostly for sinking piles below 12 feet, the manually operated one being used for the initial sinking of the piles. The piles were tapered on the thinner side; no iron shoes were considered essential as the strata underneath was of pure sand. Cast iron caps were used on the top to avoid any damage and splitting of the piles. The sinking was done by striking the heads of the piles by monkeys weighing 8 to 10 maunds. The drops normally kept were 4 feet except for the mechanically operated monkeys where a drop of 8-10 feet was kept. Sinking of piles did not present much difficulty. It was found that it was more difficult to keep them in position. Sometimes when the work was carried out during night hours, it was noted that in quite a few instances, the piles had been driven away from their alignment and had to be discarded.

For assessing the load-carrying capacity of the piles, a group of six piles was loaded gradually upto 32 tons, and the settlement curve plotted. It was observed that there was hardly any settlement under that load. Load-carrying capacity of each pile was also worked out by Major Sanders' Formula in the last stages of sinking and was found to be 2 to 3 times the designed load.

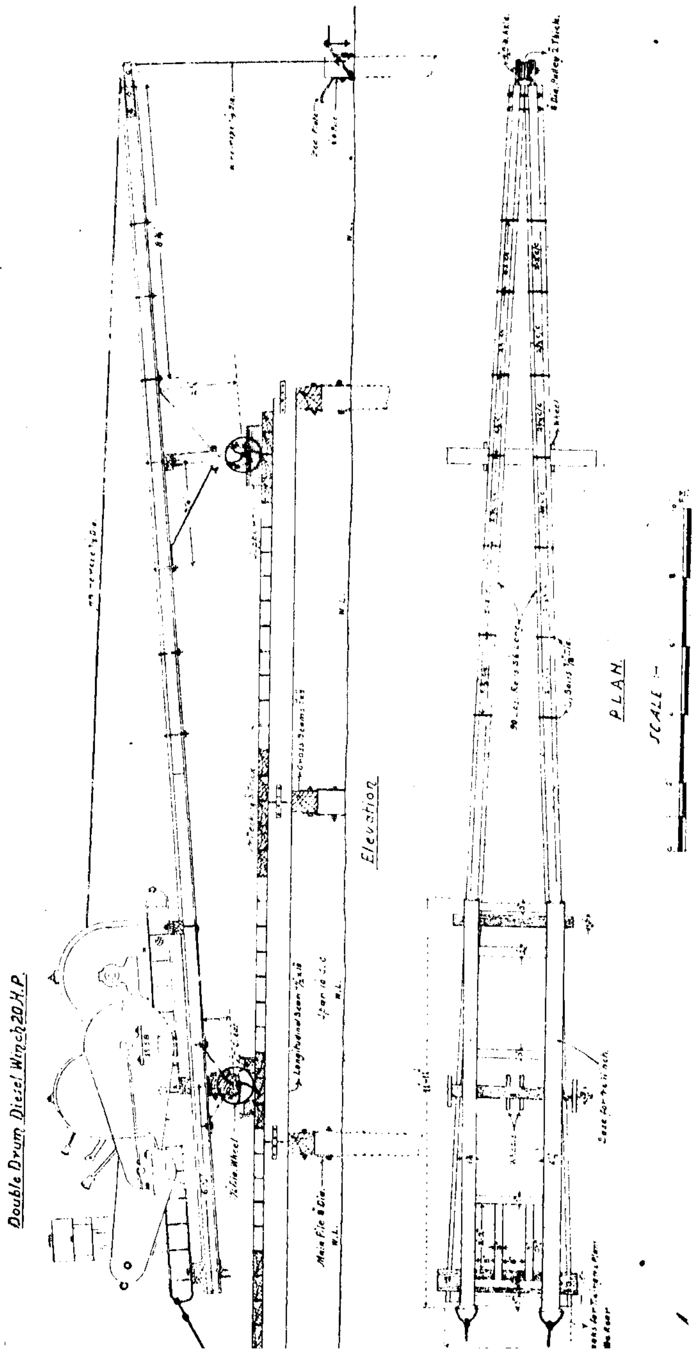
All the 450 piles for the three bridges were driven in a working period of 35 days—work on the superstructure having been done simultaneously.

Extraction of Piles—After the transport work had been completed, it became imperative to dismantle the pile bridges primarily to salvage the timber and secondly to avoid obstruction to the barges and the ferry service that is operated in the river Ghogra during the rainy season. No difficulty was experienced in dismantling the superstructure but the extraction of piles was a problem. Two winches were set up to operate together for extracting the piles which had been sunk upto 20 feet depth. One 5 H. P. winch was fitted on a pair of boats each of 200 maunds capacity. The arrangement is shown in Fig. 2. Use of two boats was essential to prevent tilting of the boats during extraction. The other winch of 15 H. P. was fitted on three wheeled trolley which was worked from the decking of the bridge. The trolley was shifted back as the piles in one span were extracted. The general arrangement of the set up of the two winches is shown in Figs. 2 and 3.

It was observed that while one single winch was not capable of extracting the piles, the two of them together were able to deliver the goods quite satisfactorily. At times, even the



PILE EXTRACTION PLANT



PILE EXTRACTION PLANT

Fig. 3

combined force of the two winches would not succeed in extracting the piles. In such cases, a little hammering on the top of the piles was found to be quite effective. Working in two shifts of 8 hours each, an average of 12 piles were extracted daily.

Unfortunately all the piles could not be extracted by the above procedure. As early as 12th of June, 1959, the water level in the river Ghogra rose considerably and it became necessary to dismantle the superstructure, thus throwing the 15 H. P. winch out of gear. Further work was carried out only by one winch fitted on the boats, but the H. P. being less, it could only extract 8 inches diameter piles in the wing walls. Having no other option, the extraction of the piles has been deferred till after the rains when attempt to extract the remaining piles would be made.

8.3. **Service Roads**—The next issue after construction of the pile bridges was the service roads which had to connect the pile bridges to the existing roads on Ayodhya and Lakarmandi sides. Roads were also necessary to be constructed around the bund for the transport of materials. In all about 4½ miles long tracks were required to be laid. Due to shortage of steel not more than 800 pierced plates could be obtained for laying tracks on sandy beds. Unfortunately even those obtained were very narrow in width being only 15 inches wide and consequently four plates had to be used for a single track, with the result that the entire number of 800 pierced plates was sufficient for laying hardly three furlong length of these tracks. As such there was no other choice left but to construct the service road tracks by laying 6 to 8 in. of boulders on the sandy beds and over-topping them with sand. To minimise the cost and quantity of boulders, only wheel tracks 2½ feet wide and 5½ feet centre to centre were constructed. Provision, of course, had been made for roundabouts and double tracks wherever necessary.

It was learnt with bitter experience that the tracks so built do not behave satisfactorily. There were too many breakages and heavy wear and tear of trucks plying on these tracks in spite of every reasonable care and precaution taken in maintaining the tracks. Several gangs of labourers with hammers and rammers had to be deputed at every furlong to break the sharp edges of the stone and get them covered and watered. The truck drivers would rather ply their trucks on bare sandy beds which had been watered than on these boulder-tracks. It was observed that the sandy tracks when watered frequently behave very well after a few laden trucks have passed over them. As water was not available everywhere, numerous hand pumps, each costing about Rs 90, were installed along the service roads. In the later stages of work, three tankers (each with a capacity of 1500 gallons) were commissioned from other divisions and were put on watering these tracks which behaved fairly well during the normal weather but there was chaos on stormy days when the high wind would cover the tracks with a foot of sand. The traffic on such days came to a standstill, till sand could be cleared from the tracks. A gang of 100 to 120 men was kept for the whole period of 4 months for maintenance of these tracks.

After this work was completed on 1st June 1959, the boulders from the tracks were salvaged and stacked on top of the bund for emergency works. Though time was very short, 90 per cent of the boulders used in the tracks were salvaged.

8.4. **Diversion of Subsidiary Channel**—Diverting a subsidiary stream of Ghogra which passed through the heart of the proposed alignment of the bund was a major technical problem, successfully dealt with after a couple of unsuccessful attempts. This stream took off at a point about 3,000 feet upstream of the bund and joined the Tehri Nala downstream. The natural width of the stream in January, when the water level was R.L. 296, was about 250 feet and the mean hydraulic depth 2.5 feet. The discharge in January was about 1500 cusecs and fell down to 900 cusecs by the end of March, when it was successfully diverted.

Looking at the stream it appeared so innocent and tame that one was tempted to block it at its narrowest neck. An attempt to do so was first made in November 1956 at the point A where the channel was hardly 170 feet wide and which according to the river condition of that time looked like the point of intake, Fig.4. It was thought that if some sort of blockade could be built expeditiously throughout the width of the channel, the flow would stop and be diverted to the main stream. Work was consequently started from both the banks. The first few feet of the bund were made with earth alone but it was observed that as the bund proceeded away from the banks, the earth dumped got all washed away. Balli crates were erected in position and were filled with sand bags but this too did not prove very effective. As the natural waterway got reduced, the scouring in the river bed became too swift. By the time 60 feet length of the stream from either side had been blocked, the depth of water in the remaining gap of 55 feet had increased to 15 feet when originally it was hardly 3 feet. The velocity of water had also increased to 8 or 9 feet per second. In the above circumstances, blocking of the gap was not possible and the attempt was abandoned.

It was reasoned by the Engineers at site that the chief reason for failure was the fact that the work was got done intermit-

tently and that blocking the channel would perhaps be possible if the crates filled with boulders were placed in the whole width of the stream expeditiously and with a bang. A second attempt to block the stream was consequently made at the point B, Fig. 4, where the water was hardly 2 feet and the flow quite controllable. Crates filled with boulders were rolled along the entire width from boats fixed on both sides. It was, however, soon observed that excessive scouring took place in the gaps left between the

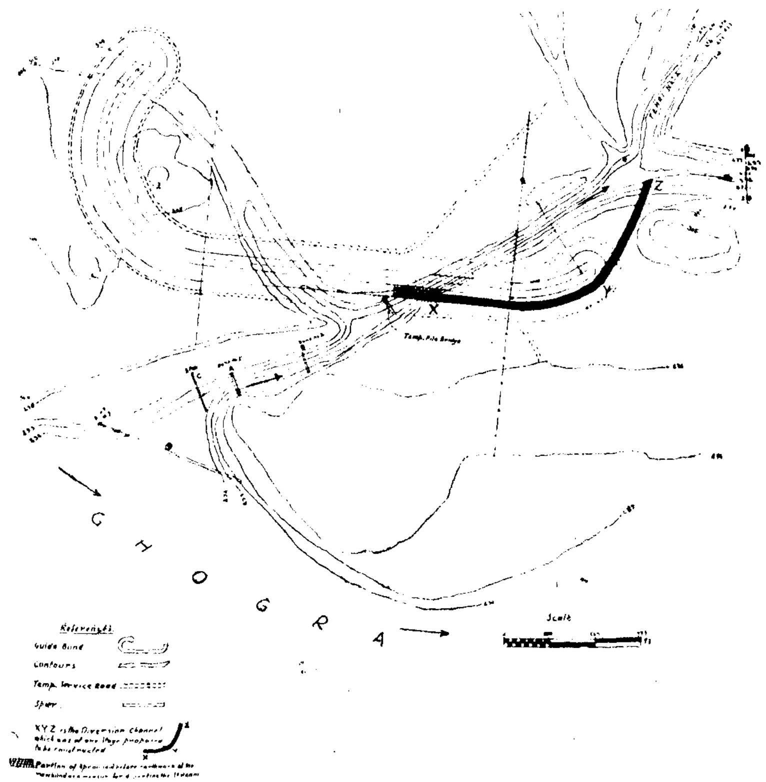


Fig. 4

Works in connection with diversion channel

two crates and when attempts to fill these gaps with sand bags were made the cages were found to sink allowing water to flow fiercely on top of them. The attempt had to be abandoned after all the 8 or 9 thousand sand bags that had been collected for the purpose were consumed and the end of the problem was not even in sight.

The above two failures had their own lessons. By now it was pretty evident that damming the stream having erodible bed would not succeed unless the discharge was reduced considerably and that better alternative would be to divert the channel without offering appreciable resistance to the natural flow of the stream. The following lines of action were consequently decided :

(i) To construct a spur at the point of intake for deflecting the stream and reducing the discharge.

(ii) To lay the apron in front of the portion of the bund which crossed the stream with a view to build a toe wall at the back of it, causing heading of water.

(iii) To construct a diversion channel across line X,Y,Z, offering alternative route to the water flowing in the stream, Fig. 4.

(iv) To construct a marginal bund 1,100 feet long across point D, 3,000 feet upstream at a time when the water level had fallen to R.L. 294.

Obviously the construction of the spur was only a partial cure to the problem. All that was intended to be achieved was the reduction in discharge so that the task of making a marginal bund or diversion channel could be facilitated. A spur was consequently made at the point C at an inclination of 120°. The top of the spur was kept 5 feet wide and 1 ft higher than the water level of 295.5 that existed in January. The first 60 feet length of spur was made purely of sand and no resistance to the water flow was encountered. After the 60 feet length had been achieved, some scouring was observed at the nose of the spur. A backing of sand bags was consequently provided on the upstream face and the earthwork carried on behind the sand bags. To retain the earth, brushwood was also put in between. By this means 100 feet length of the spur had been made, but it was soon observed that the scour at the nose was gradually increasing. Two rows of ballies 5 feet apart longitudinally and transversally were consequently sunk into the bed and bamboo mesh tied along. The space between the two rows was then filled with brushwood, jhau, sand bags and earth. The scouring became excessive as the length of the spur increased. When the spur had been extended upto 180 feet, it was observed that the scour at the nose was as much as 12 feet. Further extension of the spur without incurring heavy expenditure was not possible. The spur had, however, achieved its purpose. Partial deflection of the stream had been attained but an appreciable amount of water along the spur into the subsidiary channel, Fig. 5.

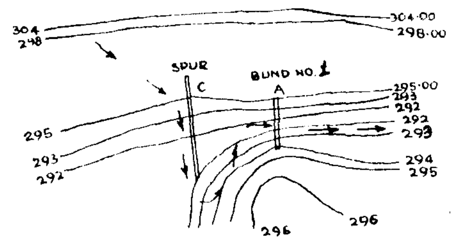


Fig. 5

While the spur was being constructed, work on laying the apron in front of the portion of the bund which crossed the stream was also started. The apron had ultimately to be laid as a part of the bund. Normally laying of apron is started after the earthwork for the main bund is done. In this case it was considered that if a 90 feet wide apron could be laid in the flowing stream, it would be possible to take help of this apron and build a toe wall at the back, and carry out earthwork behind it. The work of apron laying in this region was done piece-meal. At first a 30 feet wide strip beginning from the toe was laid. It was noticed that as the work progressed, the boulder strips sank here and there and the entire discharge of the stream passed on top of the boulders.

It had been contemplated that after this apron had been laid, a toe wall 15 feet upstream from the inner edge of the apron would be built on top of the apron to cause heading of the water and facilitate the diversion of the stream along the channel X,Y,Z.

Since none of the above remedial measures could be fully relied upon, it had also been contemplated that a diversion channel across the line X,Y,Z, would be made allowing the water to flow along the bund into the Tehri Nala. Of all the remedies, this was the surest one. The hitches about adopting this course were many. Firstly, excavating a diversion channel below the natural sand bed was a difficult task on account of the seepage water from underneath. Secondly, diverting the stream along X,Y,Z, would have necessitated quite an appreciable portion of the apron in this length to be laid in water which would have resulted in considerable extra expenditure. Diverting the river would have also necessitated change in the alignment of the service roads and would have caused a hold up in the movement of the boulder traffic which could be ill-afforded. It was because of these practical difficulties that while arrangements to construct the diversion channel were going on, steps were also being taken to construct a marginal bund 1,100 feet across the point D, 3,000 feet upstream.

Perusal of Fig. 4 would show that for such streams as these, it was difficult to ascertain correctly the point of intake. Its location depends on the water level. In January when the water level was about 296, the point of intake could be taken as point A, but when the water level went down to R.L. 294, the point of intake was obviously along the point D. Here, though the width of subsidiary channel was very wide, the depth of water was hardly 1 foot. After reconnoitring the site it was thought that if a marginal bund could be thrown across the point D, as shown in Fig. 4, the flow in the subsidiary channel could be stopped as the natural flow in the river Ghogra is not resisted. It was consequently decided to construct the marginal bund 1,100 feet long along this line in late March when the water level goes down in the river Ghogra.

The work on the marginal bund was started on the 25th of March, 1959. In the beginning, it was constructed purely with sand. As soon as some resistance was met with, the sand was replaced by gunny bags filled with sand. The sand bags were laid with break joints, headers and stretchers. It had been planned that the construction of this marginal bund with gunny bags would continue till scouring at the nose was observed as in previous cases when openings were left in the bund in the form of hume pipes laid over a layer of boulders. By allowing some discharge to pass through hume pipes, the force of the river was expected to be greatly reduced enabling the construction of the remaining length of the bund. To ensure water tightness the space between hume pipes was to be filled with brushwood and gunny bags. Insertion of such hume pipes was to be done every time scouring at the nose or blowing of the sand through the sand bags was observed. It was felt that by providing such openings, it would be possible to construct the whole length of the marginal bund, when it was to be strengthened and widened such that the line of hydraulic gradient may fall outside the embankment. The final plan was to close the mouth of the hume pipes by inserting iron discs in the sockets in the hume pipes keeping a constant vigil on the bund which might breach as a result of rise in the water level upstream. Fortunately no such resistance was met with in the first 900 feet length of the marginal bund. With gunny bags carefully laid and netted, work on extending the marginal bund went on smoothly till the last 100 feet gap was left when, in spite of precautions, the quantities of gunny bags were all exhausted. On the next day when the work was resumed, it was observed that flow of water in the gap had become pretty swift.

Fresh stock of gunny bags was again arranged and a detour made in the alignment of the marginal bund so as to avoid the newly scoured zone. This of course increased the length of the bund somewhat but had the advantage that the water depth met with was not very excessive. With a labour strength of 1,000 people and ready stock of sand bags, the work of closing the gap of 100 feet was started on the morning of 30th March. Without allowing water to scour its bed the gap was closed within a couple of hours. As soon as this was done, the difference in water level on the upstream side and downstream side increased, and the whole labour force was spread along the whole length of the bund for widening and strengthening it.

Earthwork on the main bund was also started simultaneously and continued round the clock. Within a couple of days earthwork on the main bund 2 feet above the water level had been done. During the time this was being done, vigil was kept on the marginal bund.

Success at the marginal bund was achieved just in time and work in connection with diversion channel was abandoned. No hume pipes had to be provided in the marginal bund presumably because of the great reduction in the discharge of the stream. Had the marginal bund not succeeded, a great deal of expense would have been incurred in constructing the diversion channel.

8.5. **Earthwork**—The one item that did not have to wait for the completion of the pile bridge or the service road was earthwork and it was started as soon as most of the contracts for boulder supply had been finalized. Unlike the construction of Ayodhya side bund where earth had to be brought by boats and mules, plenty of earth was available in the vicinity of the Lakarmandi side bund and was made use of.

The last two floods had changed the topography of the river bed a great deal. The first half portion of the bund on the upstream side had now to be built on quite a high ground and in fact at a R.L. of about 205, i.e., 11 feet higher than the low water level. The problem now arose as to whether the apron should be laid at the existing ground level or at the low water level as previously contemplated and as is normal in case of guide bunds. Alternative proposals considered were:

(i) To lay the apron at R.L. 294 as originally designed by excavating the bed.

(ii) To lay the apron at R.L. 299 such that the top of the apron at its outer nose when laid may remain flush with the existing ground level.

(iii) To lay the inner edge of the apron at R.L. 294 as previoulsy intended and raise the outer edge gradually to R.L. 299.

(iv) To reduce the width of the apron by half and increase the thickness by the same amount.

The pros and cons of the above four alternatives were thoroughly considered. Apart from the question of huge amount of earthwork and the cost involved therein, the time factor had also to be considered. From the considertions of cost and time, it was best to lay the apron at R.L. 299 but this idea was discarded since, by adopting this proposal, the pitching on the slopes would have had to be necessarily curtailed and, in case of scour, reliance placed on the apron to pitch up the slopes. This was not considered desirable as long as it could be helped. Proposals (iii) and (iv) also could not find favour as launching of apron is seldom uniform. The only alternative left was to lay the apron at L.W.L. The concession made here was to lay the apron at the water level existing at the time of laying instead of sticking to the theoretical L.W.L. of 294. Consequently the apron on the upstream nose was laid at R.L. 297 while at the downstream nose it was laid at R.L. 295. The total quantity of earthwork in the main bund, apron cutting and the part of Lakarmandi side approach road executed in a period of 120 days was 100 lakh cubic feet.

8.6. **Stone Supplies**—As was done for Ayodhya side bund, all the boulders required were arranged from Vindhyachal ranges and the loading stations were Shankargarh, Lohagra, Manda-Maja Road, Birohi, Vindhyachal, Chunar, Dagmagpur, and Pahara. Some stone was also brought from Jarwa and Katerniaghat quarries. Out of 46,00,000 cubic feet of boulders required for the Lakarmandi side bund, 36,00,000 cubic feet of boulders were brought from Vindhyachal ranges and about 10,00,000 cubic feet from the Jarwa quarries. Benefitting from experience gained last year, quite an appreciable quantity of boulders for the Lakarmandi side bund had been arranged beforehand in the monsoon period of 1958, with the result that when the construction of this bund was started in January, 7,43,000 cubic feet of boulders were available at the Ayodhya siding and 6,37,000 cubic feet of boulders at the Katra and Lakarmandi sidings, and arrangements had to be made only for the remaining 32,00,000 cubic feet of boulders.

This year 4,570 wagons from broad gauge and 1,500 wagons from meter gauge were required. With the active co-operation of the railways, specially the Northern Railway, the required number of wagons were received within the stipulated time, To watch over the loading and availability of wagons, departmental representatives were posted at each of the loading stations and they were asked to post daily in the forms given in Appendix 1.

8.7. **Transport of Boulders**—The pile bridge on the main river was completed, on the 10th of February 1959 and after tests and modifications it was opened to traffic on the 20th of February 1959. There were thus only 100 days to transport all the 46,00,000 cubic feet of boulders to the site. Allowing for the numerous holdups, Holi Holidays, and the Ramnaumi Fair when all the vehicular traffic on the roads is stopped by the district authorities for about a week, there were hardly 85 days to transport 46,00,000 cubic feet of boulders. As many as 140 trucks could be arranged for the work at one time, and the number of trips raised to 580 on a single day. On peak days, the quantity of boulders transported was as much as 80,000 cubic feet. The transport work was carried out round the clock and it was mostly in the night hours that most of the work was done.

For efficient transport arrangements and to avoid pilferage of material due precautions had to be taken and necessary staff had, of course, to be maintained. During the peak months as many as 100 people were engaged for checking the trucks, issuing tickets, for collecting tickets and for supervising the check points that had been scattered all along the 4 mile route from Ayodhya to the site of the bund.

It had been planned that the transport of all boulders from Ayodhya to site should be completed by 1st June, when the pile bridge was to be dismantled. Transport from Lakarmandi and Katra where the service road was at a much higher ground level was to be continued throughout June. Unfortunately, the monsoons set in too early last year. Right towards the end of May, there was a sudden rise in the water level and a major portion of the service road from Ayodhya side was inundated. All transport by road had consequently to be stopped and 25,000 cubic feet of boulders which were left behind were transported by boats.

Transport from the other sidings also could not be continued throughout June, for as early as 13th June, the water level touched the O. F. L. mark. By that time, however, work in the main bund had been completed and boulder was only required for the reserve stocks that were planned to be kept on the top of the bund. This was subsequently brought by boats.

8.8. **Lakarmandi Side Approach Road**—No provision for the approach road had initially been made in the estimate of the guide bund sanctioned by the Government of India. It was, however, felt that to avoid danger of the river outflanking its designed course, it was necessary that the approach road on the Lakarmandi side which runs through open country should be constructed at least upto the H. F. L. Work on the approach road upto the Tehri Nala where another major bridge would have to be constructed was accordingly taken in hand. For paucity of funds and short time available, earthwork in only one mile length of the road had been completed upto R.L. 311, i.e., 1 foot higher than H.F.L. As the first 500 feet portion of this approach road starting from the abutment point would be subjected to a great deal of wave action during the rains, the slopes of the earth embankment had been protected by providing a toe wall built of boulder cages and stone pitching upto O.F.L. and 'sarpat' plantation had heen provided. These measures were purely temporary, as after the formation level of the bridge had been fixed, the earthwork in this portion would have to be raised. About 33 lakh cubic feet of earthwork had been done in this approach road. Photograph 9 shows the work on construction of toe wall, stone pitching, and sarpat plantation on the approach road.

9. ORGANISATION

For carrying out the work, a new Division of the Uttar Pradesh Public Works Department, was opened at Faizabad with the following technical staff:

Executive Engineer	1
Assistant Engineers	3
Overseers	10
Computor	1
Draftsmen	2

In addition, about 120 work agents, work supervisors and mates were kept on work-charge basis at the loading stations, various check points and at the site of work.

At first, contracts for the whole work were called but as the response from the contractors was not favourable and the rates tendered were very high, it was decided to split up the work in small groups. For the construction of Ayodhya side bund, separate contracts in 6 groups were called for the supply of boulders which were to be stacked at the sidings. Separate small contracts for the transport of boulders to the site of work and laying it in apron or pitching it on the slopes were then arranged. The earthwork and the work of brick ballast con-

solidation was also split up into several small groups and given to different agencies. These arrangements had some drawbacks. Certain discrepancies were observed in the quantity of boulders collected and in the quantity of boulders laid in apron and pitching work.

To avoid the recurrence of such discrepancies, tenders for the Lakarmandi side guide bund were called for the supply of boulders, and laying it at the site. Payment was made to the contractors for the finished work. As before, the work was divided in 5 main groups for the boulder supply and laying. Earthwork was also divided into number of groups. The job of kankar pitching and brick soling was, however, given to one party.

10. COST

The total estimated cost of the work	...	Rs 80.52 lakhs.
Actual cost of Ayodhya side guide bund	...	Rs 24,34,000
-Do- Lakarmandi side guide bund	...	Rs 50,76,600

The above figures include all temporary and permanent works connected with the project. The cost per running foot of the bund works out to Rs 923 on Ayodhya side and Rs 1,061 on Lakarmandi side. Appendix 2 gives the break up of the actual expenditure.

11. CONCLUDING REMARKS

The success of a project like this, which has got to be completed in a scheduled time, entirely depends on efficient arrangements for transport of materials, careful planning, good organization and close liaison between the railways and the department concerned. The technical problems likely to be faced are: (a) doing earthwork in flowing water, (b) laying apron in deep water, (c) construction of tracks on sandy beds, and (d) diverting of subsidiary streams.

ACKNOWLEDGMENT

The Author wishes to thank the Chief Engineer, P.W.D., U.P., for his permission to publish the Paper. He also thanks Shri R.C. Sharma, Assistant Engineer and the other staff for the assistance given by them in the compilation of the Paper.

Appendix—1

PROGRESS CARD FOR LOADING STATIONS

THE EXECUTIVE ENGINEER,
TEMPY. BRIDGE CONSTN. DIVI. FAIZABAD.

Station....................

Date.......................

The progress report on wagons at the above Stations on this date is as follows.

S. No.	Name of Contractor	No of wagons Regd. on date		No. of wagons loaded on date	Total wagons loaded upto this date	Quantity available at the quarry
			2			
1.	Shree D. S. Acharya					
2.	M/s Babu Ram Ramanandi					
3.	Shree B. N. Agrawal					
4.	M/s Allied Builders					
5.	M/s Agrawal Trading Co.					

Note :—In Col. 2 please fill pending registration on date. *Sd:*........................

TICKETS FOR ISSUE OF BOULDER FROM SIDINGS

Book No. S. No............

Construction of River Training works for the Proposed Bridge over River Ghogra at Ayodhya

(Carriage of Boulders)

Truck No................ Height...............

Collected by Shri.................................

Cartage by Shri...................................

Stack No..

Signature of Driver.............................

Signature of Issuing Officer..................

Book No. S. No............

Construction of River Training works for the Proposed Bridge over River Ghogra at Ayodhya

(Carriage of Boulders)

Truck No................ Height..........

Collected by Shri............................

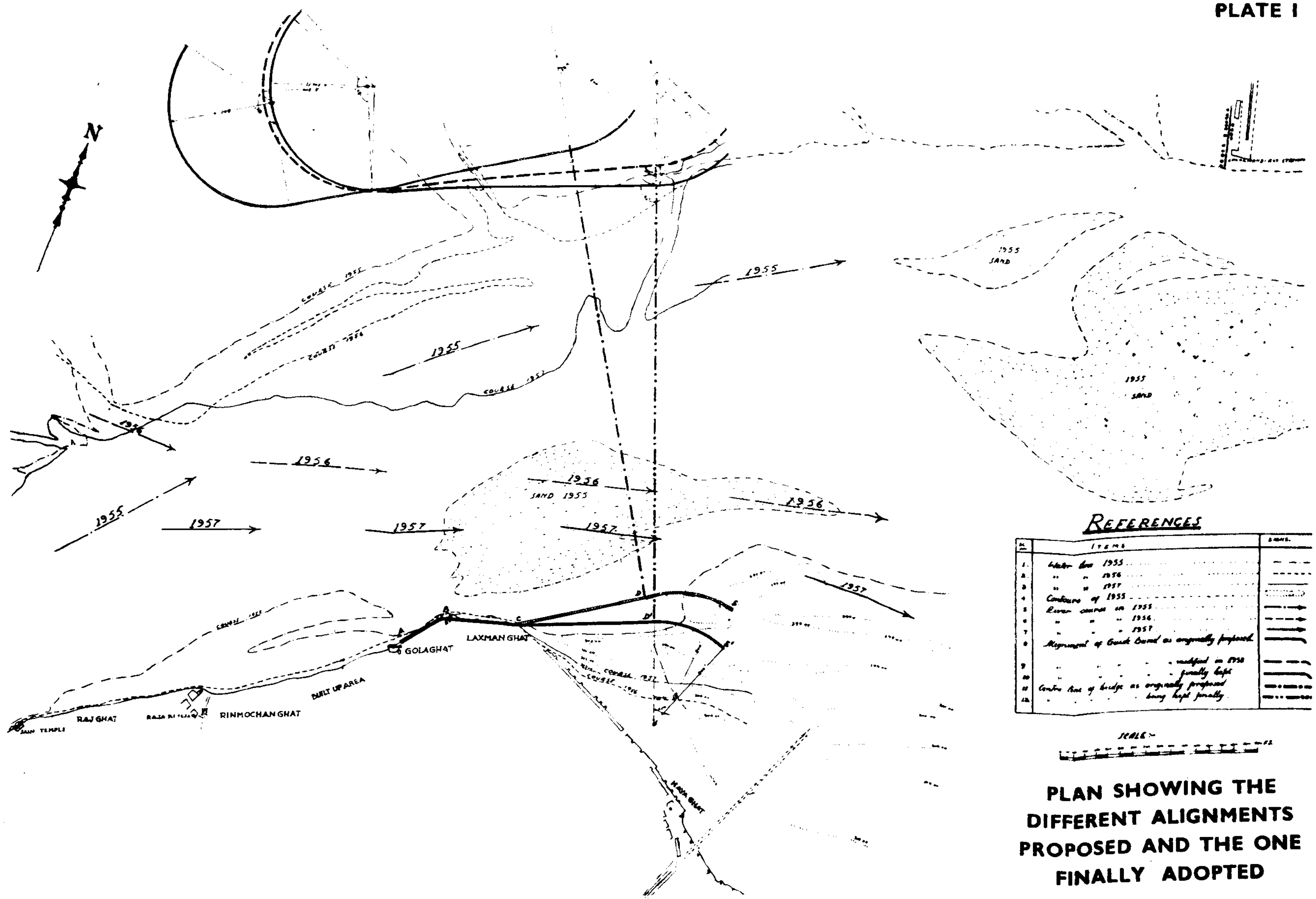
Cartage by Shri...... . ...

Stack No..

Signature of Driver.........................

Signature of Issuing Officer.............

PLATE I



PLAN SHOWING THE DIFFERENT ALIGNMENTS PROPOSED AND THE ONE FINALLY ADOPTED

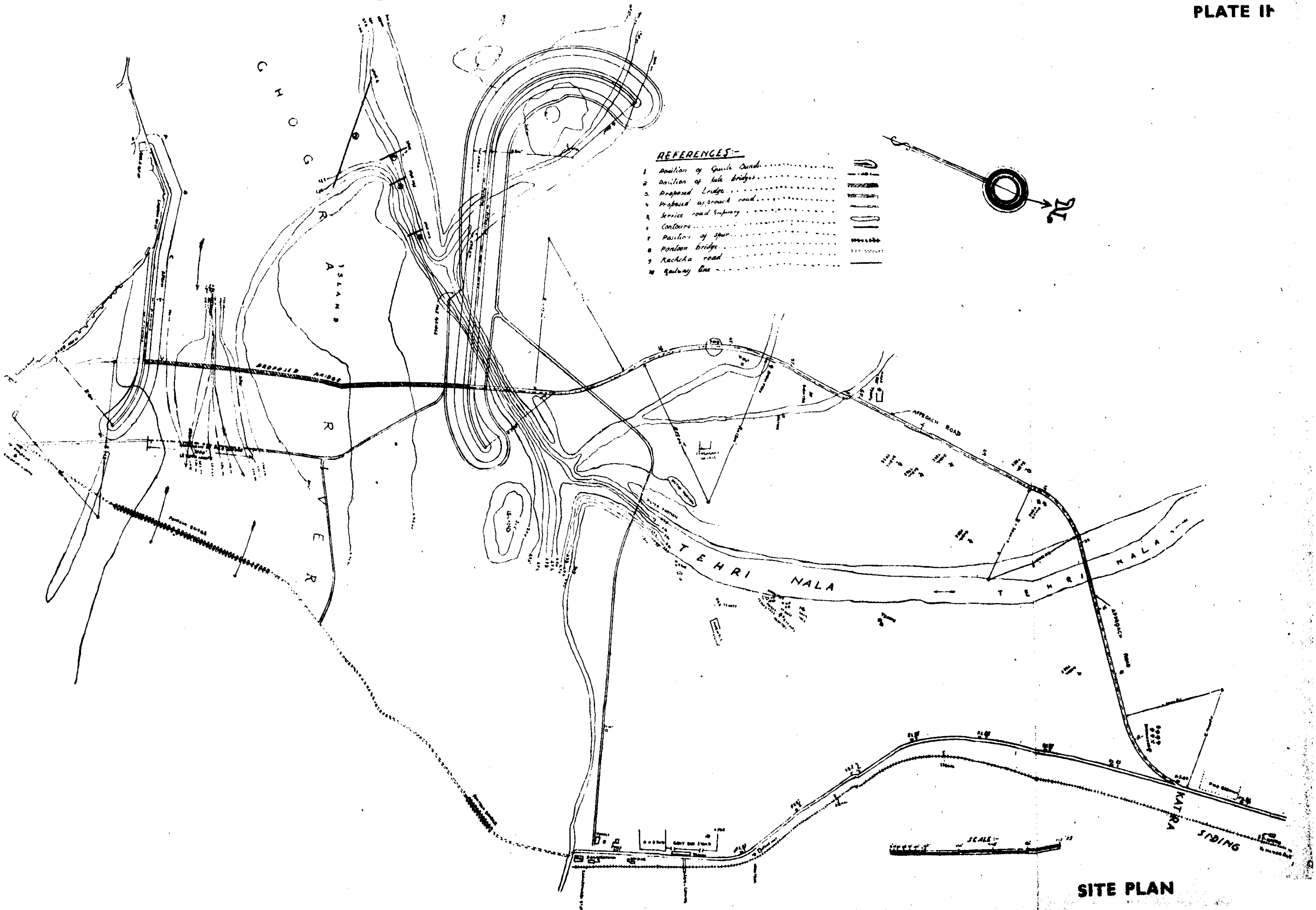
Appendix-2

Statement showing Expenditure incurred on different items of work on construction of Ayodhya and Lakarmandi Side Guide Bunds

Estimated Cost: Rs 80.52 lakhs

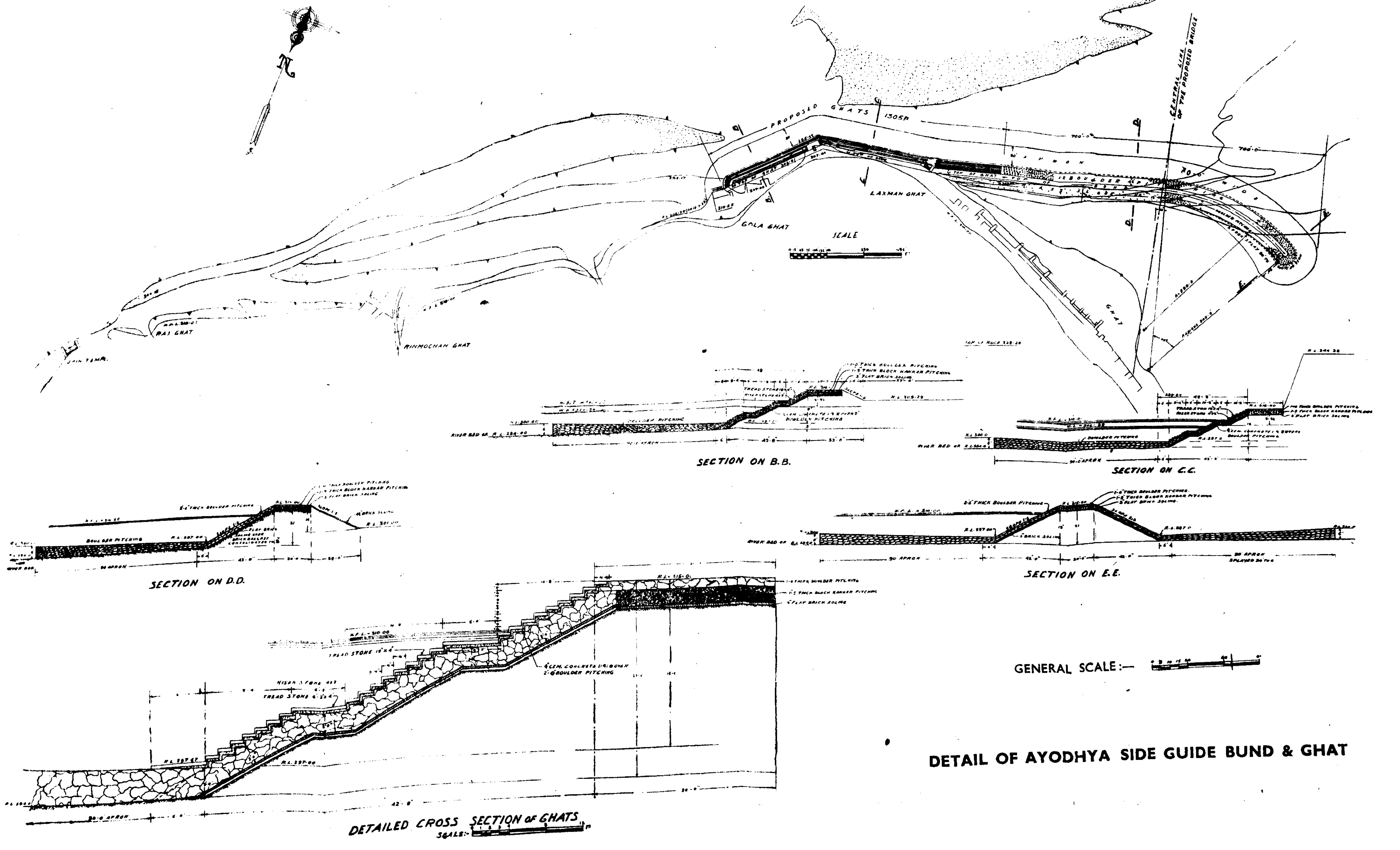
Sl. No.	Item of Work	Ayodhya Side Bund		Lakarmandi Side Bund		Grand total	Remarks
		Quantity	Cost	Quantity	Cost		
1	2	3	4	5	6	7	8
1.	Earthwork in guide bunds ...	28,54,564 cu. ft.	1,35,087	63,74,000 cu. ft.	1,47,000		
2.	Flat brick soling ...	1,62,000 sq. ft.	30,750	4,30,000 sq. ft.	58,300		
3.	Supply and consolidation of brick ballast ...	42,042 cu. ft.	18,919	1,16,900 cu. ft.	55,700		
4.	Block kankar pitching ...	77,852 cu. ft.	38,926	1,32,500 cu. ft.	68,600		
5.	Apron ...	14,01,196 cu. ft.	11,77,005	33,14,291 cu. ft.	26,74,296		
5(a).	Extra for crates ...	...	15,000	...	...		
6.	Pitching ...	4,67,407 cu. ft.	3,92,622	10,72,000 cu. ft.	10,36,800		
7.	Pile bridge (construction & extraction) ...	1,200 rft.			2,71,160		This figure is inclusive of cost of salvaged material.
8.	Diversion of channel & spurs ...				50,000		
9.	Construction & maintenance of service roads including approaches to pile bridges ...		5,000		29,500		
10.	Construction of Lakarmandi side approach road ...				55,139		
			18,13,309		44,46,495	62,59,804	
11.	Work-charge establishment ...	...	24,000	...	50,879	74,879	
12.	Miscellaneous charges such as temporary accommodation, tools and plants, lighting, etc.	...	10,000	...	15,000	25,000	
13.	Abutment wells (part sinking) ...	...	77,542	...	...	77,542	
14.	Ghat-work ...	...	2,36,207	...	...	2,36,207	
15.	Reserve stock ...	...	63,240	...	1,50,000	2,13,240	
16.	Land Acquisition and Rly. siding, Ayodhya...	...	40,000	...	60,000	1,00,000	
			22,64,298		47,22,374	69,86,672	
	Add centage charges @ 7½ per cent ...		1,69,822	...	3,54,178	5,24,000	
			24,34,120		50,76,552	75,10,672 say Rs 75.11 lakhs	
	Savings anticipated and proposed to be utilized for improvements ...	...		...	...	5,41,000	

PLATE II



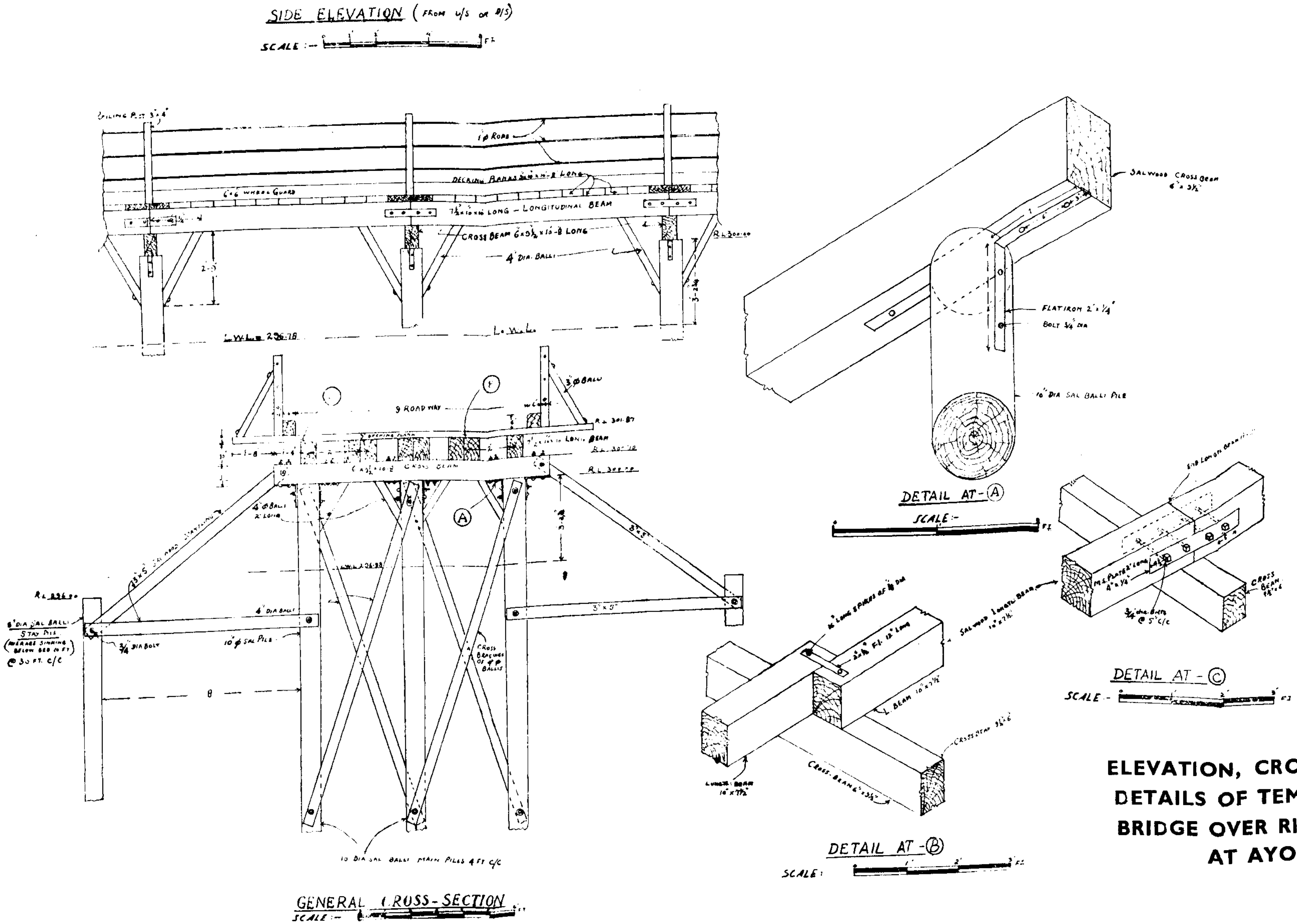
SITE PLAN

PLATE III



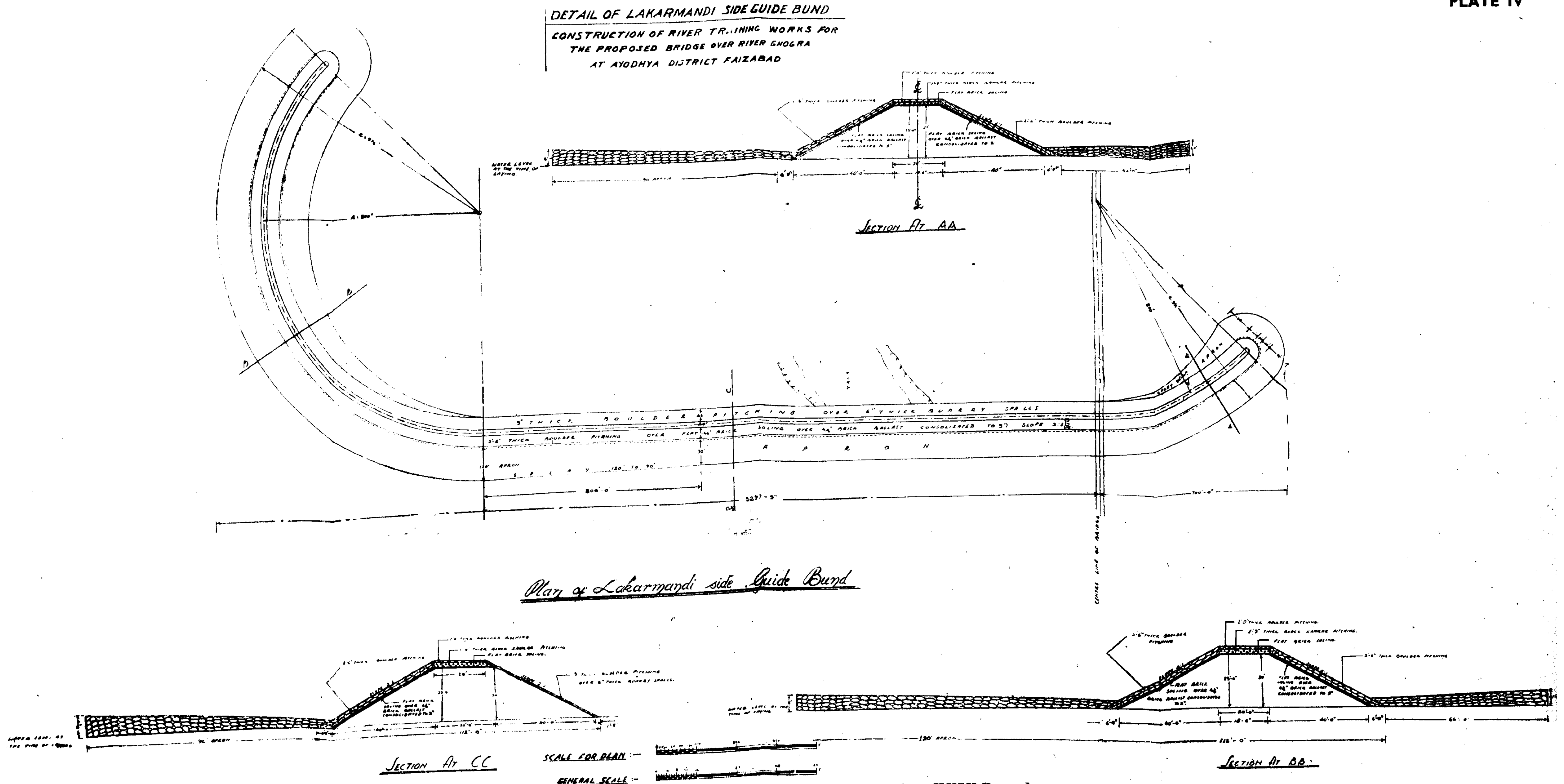
DETAIL OF AYODHYA SIDE GUIDE BUND & GHAT

PLATE V



ELEVATION, CROSS-SECTION & DETAILS OF TEMPORARY PILE BRIDGE OVER RIVER GHOGRA AT AYODHYA

PLATE IV


DETAIL OF LAKARMANDI SIDE GUIDE BUND
CONSTRUCTION OF RIVER TRAINING WORKS FOR
THE PROPOSED BRIDGE OVER RIVER GHOGRA
AT AYODHYA DISTRICT FAIZABAD
Section At AA
Plan of Lakarmandi side Guide Bund
Section At CC
SCALE FOR PLAN
GENERAL SCALE
Section At BB

Paper No. 215

"INVESTIGATION INTO CAUSES OF CRACKS IN CERTAIN CONCRETE PAVEMENTS"

By

BRIG. S. K. BOSE,* B. Sc. (ENG.), M.I.E.

CONTENTS

SYNOPSIS

The behaviour of two concrete runway pavements subjected to increasing wheel loads and tyre pressures were observed and recorded in this Paper. In one case, the observations were carried out over a period of approximately 10 years. The behaviour of the old runway pavement over a period of eight years is first discussed with reference to field data. Calculations of stresses due to wheel loads and temperature stresses are given. The old pavement was then overlaid with a new layer of concrete pavement after replacing the damaged slabs. The behaviour of the concrete overlay over a period of two years is recorded.

In the second case, the observations were made over a shorter period. One interesting feature in this case, however, was that an experimental stretch of the runway was overlaid with a bituminous pavement.

The behaviour of both the bituminous and concrete overlays at Hakimpet is also recorded.

1. INTRODUCTION

This Paper records the observations on behaviour of concrete runways at Poona and Hakimpet when subjected to

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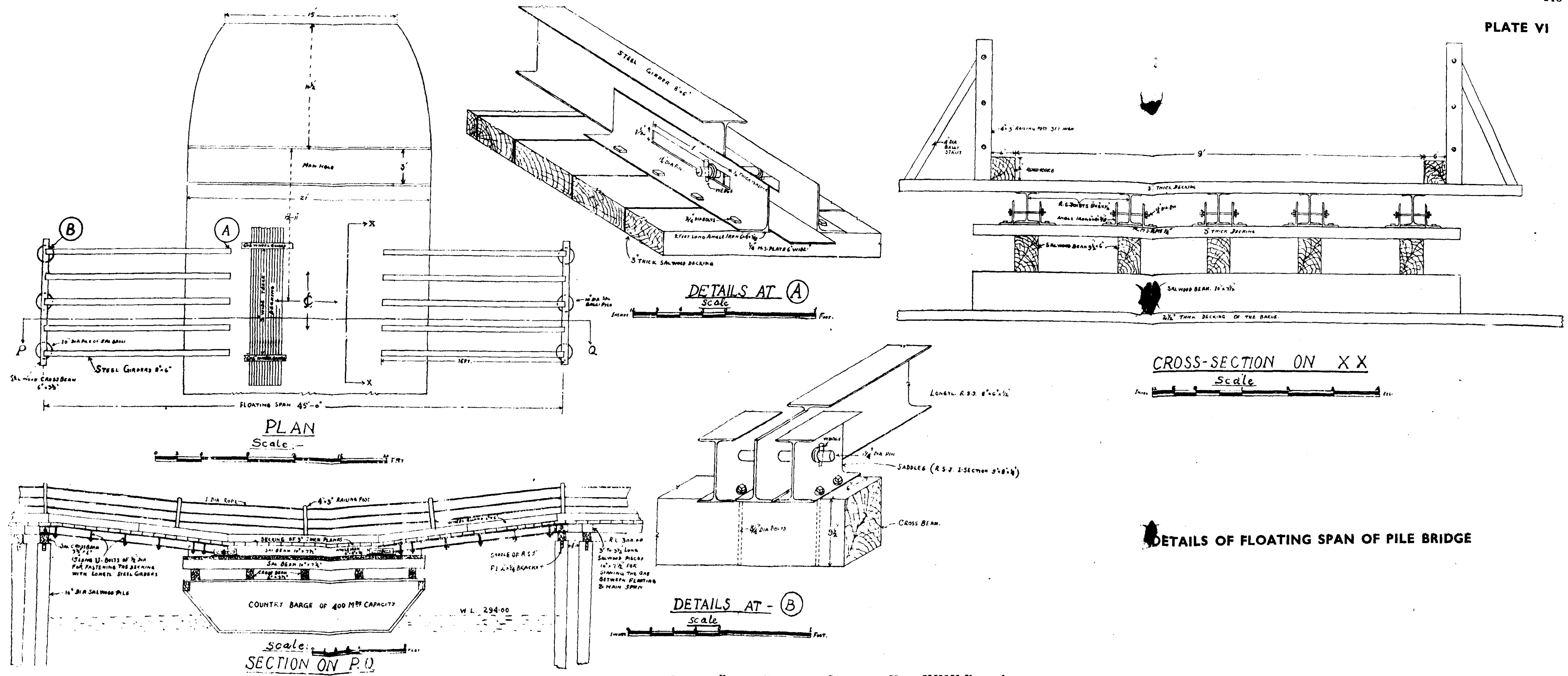
PLATE VI



DETAILS OF FLOATING SPAN OF PILE BRIDGE

increasingly high wheel loads and tyre pressures. The observations were made first on the concrete runways and later on the overlays.

For convenience, the Paper has been divided into five parts, namely, behaviour of the concrete runway pavements at Poona, behaviour of a concrete overlay, behaviour of the concrete pavements at Hakimpet, behaviour of a bituminous overlay over an experimental stretch and behaviour of a concrete overlay over the runway.

2. POONA RUNWAY PAVEMENT

2.1. The runway at Poona is situated on an elevated ground approximately 1,930 feet above the mean sea level. The site has a good natural slope and the subgrade is well-drained. The runway lies in easterly and westerly directions and the width of the concrete pavement is 50 yards.

2.2. Subgrade

The subgrade consists of moorum which is a decomposed rock. In evaluating the strength of the concrete runway it was necessary to determine, apart from the properties of the concrete pavements, the value of the reaction of the subgrade, i.e., the value of 'K' which is known as the Modulus of Soil Reaction. This is defined by the slope of the stress-deformation curve obtained by field bearing tests which were adjusted for saturated subgrade conditions.

2.3. Modulus of Soil Reaction

In more detail the modulus of soil reaction 'K_u' with subgrade at or near its optimum moisture, is determined by the formula: $K_u = P/0.05$, in which 'P' is the load intensity in pounds per square inch causing a deformation of 0.05 inch as determined from the stress deformation curve. The stress deformation curve is usually not a straight line and since the modulus of soil reaction is defined as the slope of a chord extending from the origin to a given load intensity on the curve, the value of K_u varies with load intensity. It has been found from results of many tests that the most representative value for concrete pavement design is obtained by using the load intensity causing a deformation of 0.05 inch, and so this particular value is used in the Westergaard Analysis.

2.3.1. Apparatus used

(1) Two trucks capable of supplying a rear axle load of 15 tons each.

(2) A loading platform. (The platform is fitted with stub axles and wheels for touring).

(3) 50 ton hydraulic jack fitted with suitable gauge to register actual load.

(4) Three dial gauges, capable of being fitted with the jack by means of a frame.

(5) Steel plate of 30 in. diameter.

2.3.2. Procedure of test

The tests were carried out on runway shoulders and in one case by breaking the existing concrete pavement and base course and carrying out the test on the soil. Dry sand is spread on top of the subgrade, and the 30 in. dia. plate is bedded on top of it. The plate is to be bedded as level as possible.

The three dial gauges are set on the plate equally spaced on the periphery, with their spindles almost at top of travel. A lash of about 20 thousandths of an inch should be allowed to ensure complete movement of the pointer.

A load of approximately 5 lb per sq. in. is then applied before commencement of the test to ensure even bedding of the plate in the subgrade.

The load is released and the dials are set at zero reading; successive load increments of 5 lb per sq. in. are then applied to the maximum load available, and the dial readings are taken for each load increment. Sufficient time for each load increment to produce maximum plate displacement is allowed.

The average of the three dials is taken as the final displacement for each load increment.

The readings are then graphed (load per sq. in. on plate against displacement of plate in inches) and the values of K_u obtained for any given deflection.

$$K_u = \frac{\text{Load per sq. in. on plate in lb per sq. in.}}{\text{Displacement of plate in inches.}}$$

The maximum displacement of plate which may be used in this calculation is 0.05 in. and wherever possible loads should be applied to produce this displacement.

After the maximum load increment has been applied, the load is completely released and a minimum of 5 minutes allowed before dial readings are taken.

The difference between this average reading and the average reading for the maximum loading is then taken and the result calculated as percentage recovery.

Average displacement at max. load, say,		0.05 in.
do	zero ,, ,,	0.01 ,,
	Difference	0.04 ,,

$$\text{Per cent recovery} = \frac{0.04}{0.05} \times 100 = 80 \text{ per cent.}$$

The whole procedure is repeated at least twice or three times without removing the plate on the same place depending on the results obtained. If between the first and second applications of the load, the difference in deflection at maximum loading is less than 0.02 in., then two applications are sufficient. If not, a third application is required.

2.3.3. Correction for saturated subgrade

After the value of K_u is determined, a correction is applied to allow for the decrease in bearing strength due to possible saturation of the subgrade. This reduction in bearing value is obtained from a comparison of standard consolidation tests on the soil in the optimum moisture condition and in a saturated condition. The C.B.R. test may also be used for this purpose. The formula then becomes :

$$K = \frac{P_s}{P} \times K_u \text{ in which } K \text{ is the}$$

Modulus of soil reaction for saturated subgrade, K_u determined as above, P is the load intensity used to determine K_u and P_s is the load intensity required on saturated soil to produce the same deformation as P in the original test.

2.3.4. Photos 1 to 4 show the apparatus used for this purpose.

2.3.5. The results of the evaluation of K_u are shown in Table 1.

TABLE 1

Showing the K values of the subgrade soils

Test point No.	1	2	3	4	5	6	7	8	9	10	11	Averages
'K_u' value lb/sq. in./in	180	180	200	220	230	270	270	280	300	380	400	260
Percentage recovery	60	45	37	34	29	48	40	36	38	37	27	40
'K_u' value lb/sq. in./in. after compaction	360	390	480	520	860	500	530	690	680	1070	1390	680
Percentage compactability	100	116	140	136	274	85	96	146	126	181	247	150

· These tests were carried out during the month of June 1948 when the subgrade was in a dry state. Test No. 6 was carried out on the subgrade after breaking up the concrete pavement and base course. Since this value was not appreciably different from the mean K value obtained and since the average rainfall in Poona is 24 inches, the average K_u alue given above was accepted as equal to K value. The subgrade showed a reasonable load bearing strength which could be increased much higher by compaction.

2.4. Poona Runway Pavements

The concrete pavement was constructed by the State P.W.D. in 1944. It has 6 in. stone base course and 4 in. thick concrete slabs laid in panels of 30 ft × 10 ft. At the time of the determination of K_u values, the concrete runway was in good condition, although no large scale maintenance had been carried out on this airfield up to the time. The general slope of the plateau upon which the runway has been built is good and provides a good natural drainage for storm water.

2.4.1. Strength of concrete

No record of the compressive strength of concrete laid in various panels was available. For the purpose of evaluating the strength of concrete used in the pavement, portions of broken slabs were cut into pieces of suitable sizes with a diamond saw machine and then tested for compressive strength. The results are given in Table 2.

TABLE 2

Showing the compressive strength of the runway slabs

Cube No.	Mean area sq. in.	Weight lb.	Weight oz.	Load	Compressive strength lb/per sq. in.	Compressive strength of a 6 in. cube
1	25.8	11	8	57	4940	4850
2	17.1	6	$1\frac{3}{4}$	48	6300	6050
3	17.15	6	7	51	6650	6380
4	17.99	6	1	53	6610	6350

Assuming the water cement ratio 0.6, the strength of 6 in. cube at 28 days would have been 3,500 lb per sq. in. as obtained from the standard graphs.

If the compressive strength is taken as 3500 lb per sq.in., the flexural strength may be taken 1/8th of the compressive strength or 450 lb per sq. in. With a factor of safety of 1.5, the safe allowable stress is 300 lb per sq. in.

2.4.2. **Stresses due to wheel loads**

The design data for the existing runway pavement were not known. The pavement developed cracks in several concrete slabs after it was subjected to considerable traffic having single wheel load of 37,500 pounds and tyre pressure of 75 pounds per sq. in. The stress due to wheel loads is worked out and compared with the safe working stress of the concrete determined in para 2.4.1.

Single wheel load P = 37,500 lb

Tyre pressure p = 75 lb per sq. in.

Radius of equivalent contact area 'a' $= \sqrt{\dfrac{P}{\pi\, p}}$

= 12.6 inches

Radius of relative stiffness $l = \sqrt[4]{\dfrac{E \times h^3}{12\,(l-\mu^2) \times K}}$

$= \sqrt[4]{\dfrac{3 \times 10^6 \times 64}{12\,(l-0.15^2) \times 250}}$

= 16 inches

where μ = Poisson's ratio of concrete = 0.15

E = Modulus of elasticity of pavement = 3×10^6 lb per sq. in.

h = Thickness of pavement = 4 inches

K = Modulus of soil reaction = 250 lb per sq. in.

Using Westergaard's formula:

b = radius of resisting section $= \sqrt{1.6\,a^2 + h^2} - 0.675h$ = 13.7 inches

As 'a' is greater than $1.724h$, $b = a$ = 12.6 inches

Therefore, l/b = 16/12.6 = 1.27

Si = maximum stress in pounds per square inch for interior loading

$$= \frac{0.316P}{h^2}\left[4\log_{10}\left(\frac{l}{b}\right) + 1.0693\right]$$

= 760 lb per sq. in. which is more than twice the value of allowable stress shown in para 2.4.1.

If a lower K value is assumed, the values of Si are given below:

K	250	200	150	100	lb per in^3.
Si	760	830	945	1045	lb per sq. in.

The variation in K values affects the wheel load stress considerably but the central warping stress is not appreciably altered.

2.4.3. **Stresses due to temperature differential**

The runway concrete slabs when subjected to single wheel loads of 37,500 pounds and tyre pressure of 75 pounds per sq. in. were stressed much beyond the allowable value for the concrete used in the slabs many of which showed considerable cracks. Moreover, additional stress due to temperature came into play. The maximum diurnal temperature variation in Poona is 28°F. The amplitude of surface air temperature variation is 14°F. The amplitude for surface temperature of concrete is $14 \times 1.6 = 22.4$°F. For a 4 inch concrete slab, the ratio of temperature differential and temperature amplitude is 0.7.* Therefore, temperature differential $= 22.4 \times 0.7 = 15.7°F$. Assuming e = thermal coefficient of linear expansion of concrete $= 3 \times 10^{-6}$ per degree F.

E = Modulus of elasticity of concrete $= 3 \times 10^{6}$ lb per sq. in.

The warping stress in the interior of the slab according to Westergaard formula $= \dfrac{E\, et}{2} \times \dfrac{(Kw_1 + \mu\, Kw_2)}{1-\mu^2}$

$$= \frac{3\times10^{6}\times3\times10^{-6}\times15.7}{2} \times \frac{1.05 + 0.15\times1.1}{1-(0.15)^2}$$

$= 87$ lb per sq. in.

Where $Kw_2 = 1.05$ and $Kw_1 = 1.1$. These two values are obtained from warping stress coefficient curve based on Bradbury's formula.*

The concrete slabs at the time of observation of cracks were subjected to fatigue caused by repetition of loads and reversal of stresses due to temperature changes.

2.4.4. The incidence of cracks

The end portions of a runway where the aircrafts stop before taking off or after landing are subjected to full effects of dynamic wheel loads. In the middle portion of a runway the wheel load effect is not fully transmitted to the pavement as the aircrafts remain partially airborne even at touch-down points.

The cracks observed in the runway at Poona appeared more towards ends of the runway whereas the middle portion was comparatively free from cracks. The incidence of cracks in various slabs from easterly and westerly ends of the runway is shown in Plate I.

The size of the slabs and the design and spacings of various joints are shown in Fig. 1. The patterns of cracks in some of the badly damaged slabs are shown in Figs. 2 & 3.

The patterns of cracks reveal that in majority of cases these are in transverse direction. They are caused due to downward deflection produced by the combined effect of wheel load and dead load of the slab and due to temperature conditions

* See Figs. 14 and 15 at pages 523 and 524 of I. R. C. Paper No. 179—by the Author published in the Journal of the I. R. C., Vol. XIX-Part 3, June 1955.

Photo 1

Photo 2

Photo 3

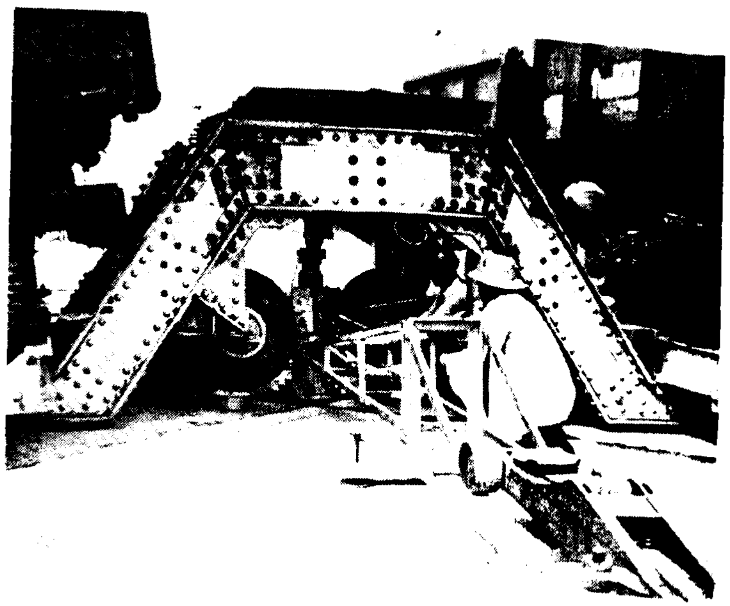


Photo 4

whereby the slab warps downwards. Since all slabs were not cracked, the upward deflection of slabs due to warping causing tension is not enough to cause the development of cracks.

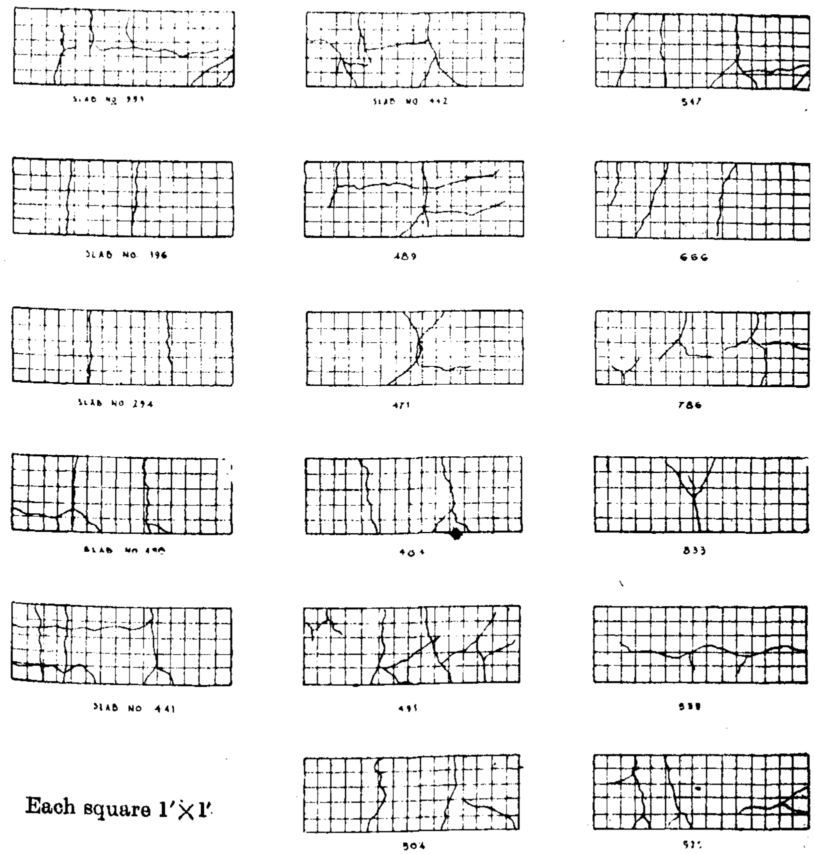


Fig. 1. **Pattern o cracks in original slabs of main runway from western end**

The diagonal cracks were due to corner loading conditions which gave tension in the top of the slab more than the allowable value due to subgrade conditions.

2.4.5. **Subgrade conditions under cracked slabs**

Although K_u values were determined in 1948, it was considered necessary to determine subgrade conditions under some of

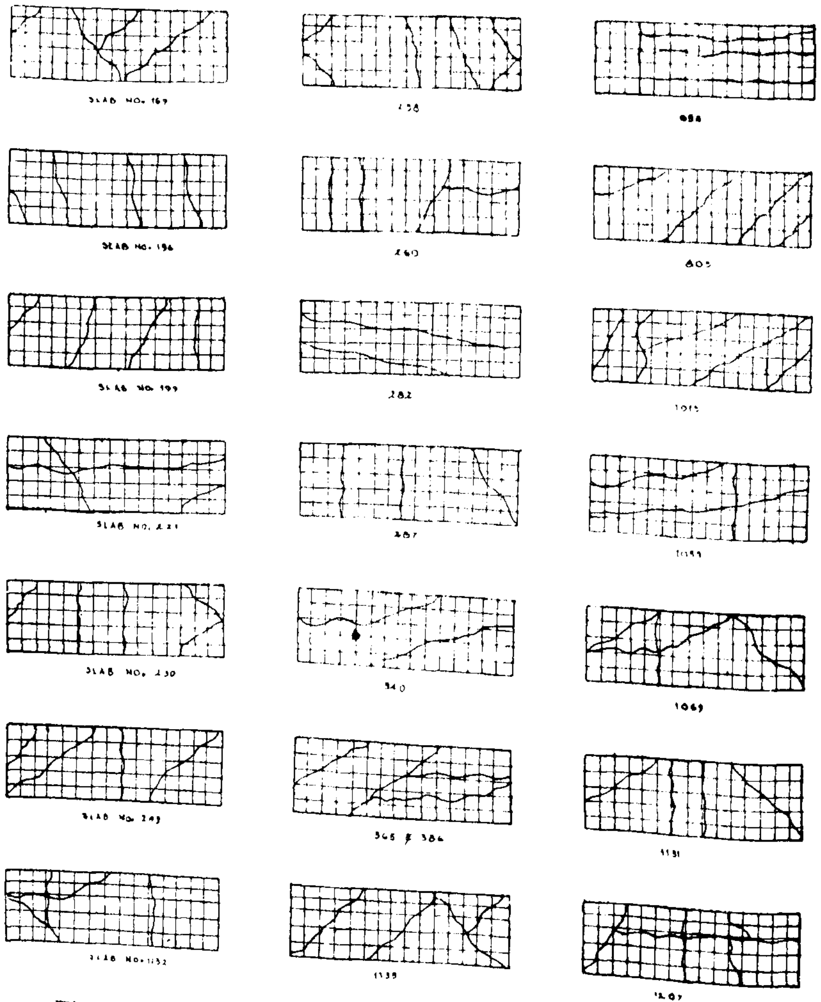


Fig. 2. **Pattern of cracks in original slabs of main runway from eastern end**

the badly cracked slabs before they were replaced. As the plate bearing test apparatus with the loading frame was not available when the cracked slabs had to be replaced, in-situ C.B.R. tests were taken and were used for determining approximate value of '*K*' from the charts published by Portland Cement Association, U.S.A. During 1955, eleven in-situ C.B.R. tests were carried out on the subgrade after cracked slabs had been removed. In-situ

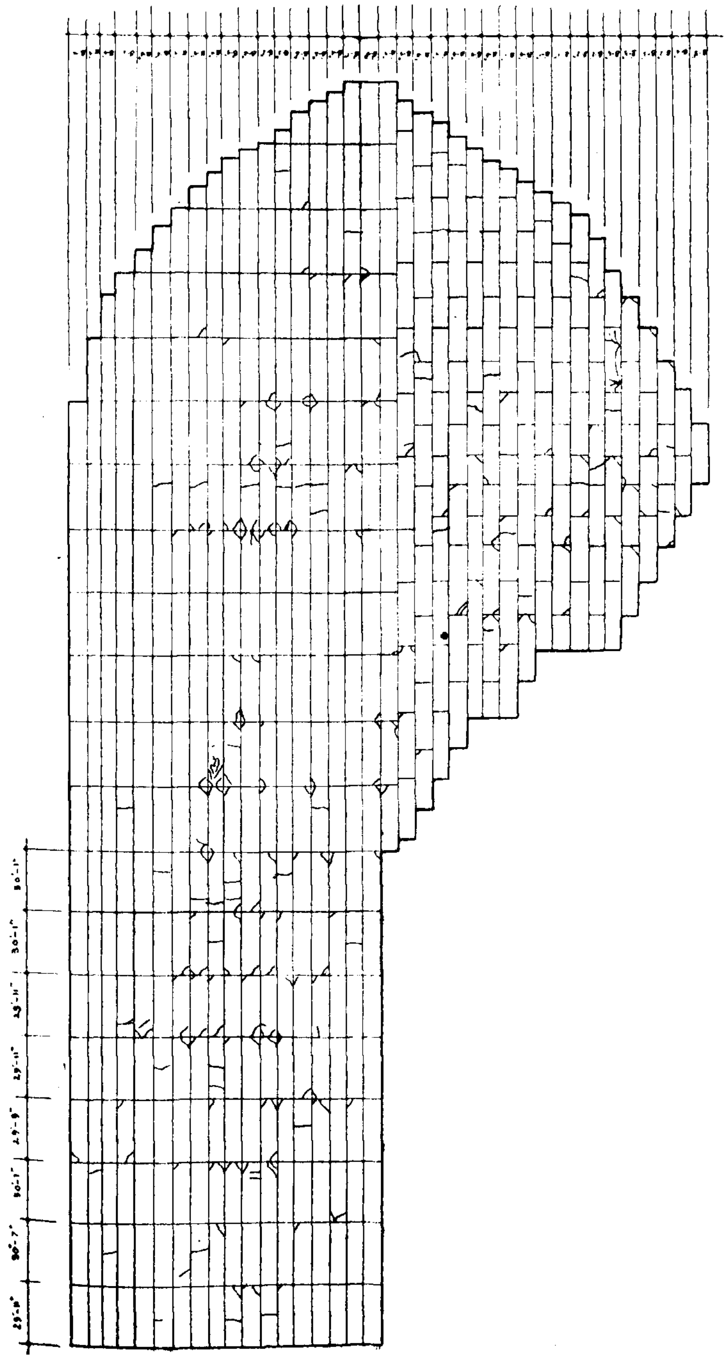


Fig. 3. **Details of old runway pavement slabs at Hakimpet showing the position and pattern of cracks from western end of runway**

C.B.R. values of subgrade under cracked slabs and the moisture content found beneath the slabs are recorded in Table 3.

TABLE 3

Showing in-situ C.B.R. values and moisture contents under some of the badly cracked slabs

Location of slabs	In-situ C.B.R. Test No.	Moisture content per cent	In-situ C.B.R. value
No. 922 from western end	1	12.5	19.7
	2	11.2	22.9
	3	14.3	21.3
No. 1059 from western end	4	13.0	26.3
	5	14.1	22.4
Near the middle of the runway but about 30 ft away from the edge	6	11.2	Test was done in corner. Result appears to be erroneous.
	7	13.2	20.4
On the centre line of runway in the middle	8	14.1	12.2
	9	14.2	13.6
	10	12.2	27.6
Near the middle of the runway about 30 ft from the other edge	11	14.1	22.8

A C.B.R. value of 20 gives an approximate K value of 250. The average K value obtained from Table 1 is 270. It would, therefore, be observed that the subgrade conditions under broken slabs were not appreciably different from the assumed 'K' value taken in the calculations.

3. CONCRETE OVERLAY

3.1. In 1956 it was decided to strengthen the runway at Poona for heavier load and also to extend it. The extension was done by laying plain concrete slab 11 in. thick in panels of five

feet squares. But the strengthening of the existing runway was carried out by superimposing on the existing concrete pavement a 6 in. concrete overlay. The panels of overlay slab weres 30 feet long and 10 feet wide. The size adopted was the same as existed in the runway pavement. The expansion jonts were, therefore, at the same spacings as already existing in the runway. This was done to suit the convenience of construction. Moreover, it was presumed that if overlay slabs had expansion joints at spacings different from the existing pavement, cracks in the overlay slabs might develop along the line of existing expansion joints. The expansion joints in the overlay slabs were staggered by 6 inches, the staggered area being treated with two layers of bituminous coating before casting the overlay slabs. This measure was taken to provide an adequate safeguard against seepage of water through expansion joints to the subgrade.

3.1.1. The contract for concrete for the overlay laid down the following specifications which were observed throughout the work:

(a) The average ultimate compressive strength of concrete shall not be less than 2,700 pounds per sq. in. (cube) at 7 days age and shall not be less than 4,000 pounds per sq. in. at 28 days age of normal curing.

(b) The water-cement ratio shall not exceed 0.5 by weight.

(c) 3 cubic feet of concrete shall be used for casting 6 in. cube specimen from each day's pouring.

(d) 6 in. core specimens shall be drilled from completed works after 28 days age and the individual strength of the core shall not be less than 3,200 pounds per sq. in. (modified to cube strength of 4,000 pounds with a correlation factor of 0.8.).

The contractor was required to design the mix to satisfy the specifications given above. The mix proportions adopted were 1.25 : 2.5 : 4.5.

3.2. After completion of the work in May 1957, the overlay pavement was subjected to single wheel loads of 24,000 lb and tyre pressure of 140 lb per sq. in. During the period June to September 1959 certain cracks were observed in some panels of overlay slabs. The date of casting the slabs and the dates of noticing the cracks as well as the strength properties of the concrete in compression are shown in Table 4.

TABLE 4

Showing details of overlay slabs which developed cracks

Slab No.	Date of casting the slabs	Date of noticing the cracks	Compression tests lb per sq. in. 7 days	28 days
A. From westerly end of the runway				
24	25- 2-57	12- 7-58	3268	5808
26	15- 2-57	20- 9-58	—	—
28	15- 2-57	5- 6-58	3268	5805
83	10- 1-57	12- 7-58	3162	4095
114	10-12-56	18-10-58	2765	4239
158	3- 1-57	20- 9-58	3459	4686
168	8-12-56	12- 7-58	2668	4299
171	2- 1-57	18-10-58	4167	5567
577	13-11-56	5- 6-58	3483	4338
587	18-12-56	5- 6-58	—	—
607	14-11-56	20- 9-58	3089	4530
624	15-11-56	23- 8-58	3195	5083
646	25- 1-57	18-10-58	3231	4589
654	29- 1-57	12- 7-58	3289	4886
714	20-11-56	18-10-58	3559	4840
915	25- 1-57	5- 6-58	3231	4589
967	26-11-56	23- 8-58	3262	5359
997	18- 1-57	23- 8-58	3214	4789
1315	8- 1-57	12- 7-58	3383	5004
1450	19- 1-57	18-10-58	3041	4736
1950	12- 4-57	23- 8-58	—	—
1953	12- 4-57	23- 8-58	—	—
B. From easterly end of the runway				
100/1	2- 4-57	23- 8-58	3704	8755
97/1	25- 3-57	23- 8-58	3328	4986
80/1	2- 4-57	12- 7-58	3704	4755
82/1	27- 3-57	12- 7-58	3334	4648
67/1	23- 3-57	8- 6-58	3348	4413
55/1	22- 3-57	12- 7-58	3210	5066
37/1	23- 3-57	20- 9-58	3348	4413
36/1	21- 3-57	20- 9-58	3196	4612
8/1	21- 3-57	23- 8-58	3196	4612
82	29- 1-57	12- 7-58	3289	4886
33	7- 2-57	20- 9-58	3427	4599

Slab No.	Date of casting the slabs	Date of noticing the cracks	Compression tests lb per sq. in. 7 days	28 days
B. From easterly end of the runway—(*contd.*)				
85	9- 2-57	20- 9-58	3383	4857
99	28- 1-57	23- 8-58	—	—
112	30- 1-57	12- 7-58	2896	4890
187	26- 3-57	12- 7-58	2906	4274
217	2- 2-57	18-10-58	3435	5316
247	4- 2-57	5- 6-58	3051	5832
263	5- 3-57	13- 8-58	3576	5297
277	5- 3-57	12- 7-58	3576	5297
495	7- 3-57	5- 6-58	3830	5208
505	9- 3-57	18-10-58	3321	4952
551	22- 2-57	12- 7-58	—	—
555	14- 3-57	20- 9-58	3486	5188
562	21- 2-57	18-10-58	3217	5031
565	10- 3-57	12- 7-58	3431	4761
567	8- 3-57	20- 9-58	3383	4927

3.3. After the observations recorded in Table 4, some more slabs developed cracks. The locations of these slabs and other details are shown in Table 5.

TABLE 5

Details of overlay slabs showing cracks after those observed and recorded in Table 4

Slab No.	Date of casting	7 days	28 days
A. From westerly end of the runway			
38	10- 1-57	3762	4095
444	11-11-56	3234	4233
487	12-11-56	3085	4382
1154	4- 1-57	3234	4454
B. From easterly end of the runway			
86/1	25- 3-57	3328	4986
11	26- 3-57	2906	4274
12	26- 3-57	2906	4274
38	26- 3-57	2906	4274
40	27- 3-57	3334	4648
276	7- 3-57	3930	5208

3.4. Stress Due to Wheel Load

The average value of K was determined by several plate bearing tests, Table I. For ease of calculation K was taken as 250 pounds per in. cube. The stress caused by a single wheel load on the original runway pavement is determined in para 2.4.2. The safe load that the overlay can take has then to be determined.

The thickness of the concrete overlay is six inches. The average crushing strength of the concrete is 4,800 pounds per sqaure inch. The flexural strength could be assumed equal to 1/8 of the crushing strength or 600 pounds per sq. in. With a factor of safety of 1.5, the safe allowable stress is 400 pounds per sq. in.

The overlay pavement was subjected to a pressure of 140 pounds per sq. in. The radius of equivalent contact area 'a' was 7.4 inches.

The modulus of elasticity of overlay, and the pavement is denoted by E_1 and E_2 respectively and is assumed to be the same, i.e., 3×10^6 pounds per sq. in. The value of modulus of elasticity of the subgrade E_s is derived from the value of 'K' already obtained.

Using Burmister's equation for a rigid circular loaded area, the settlement 'w' of the plate under a load 'P' where 'a' denotes the radius of the plate (30 in. diameter) is :

$$w = 1.18 \frac{p \times a}{E_s} *$$

$$\text{Since } K = \frac{P}{w} = \frac{p \times E_s}{1.18 \times p \times a} = \frac{E_s}{1.18 \times 15}$$

Therefore $E_s = K \times 1.18 \times 15$ or $250 \times 1.18 \times 15 = 5312$ lb per sq. in.

Assuming that the state of stress in the layered system is the same as in a homogeneous system satisfying Bousenesq equation:

$\frac{E_s}{E_2} = \frac{1}{564}$ Applying this value to Burmister's graph of settlement co-efficient, we get the value of Burmister's settlement co-efficient $F_w = .27$ for h/a ratio of 0.54, where h is the thickness of the original pavement, i.e., 4 inches and a is 7.4 inches.

* Please see pages 510, 511, 540, 541 and 542 of IRC Paper No. 179—"Influence of a superimposed layer in increasing the load bearing capacity of a rigid pavement" by the Author, Journal of the IRC, Volume XIX-Part 3, June 1955.

If E is assumed as a homogeneous layer which is eqpivalent to the two layers having moduli of elasticity as E_2 and E_s, then $E = \frac{E_s}{F_w} = \frac{5312}{.27} = 19{,}700$ lb per sq. in.

While considering the case of the overlay as a pavement lying over a subgrade having modulus of elasticity of 19,700 pounds per sq. in. (instead of the subgrade modulus value of 5,312 pounds per sq. in.), the value of K will be modified:

K modified has been determined thus:

$$K \text{ modified} = \frac{E}{1.18 \times a} = \frac{19700}{1.18 \times 15} = 1113 \text{ lb per in.}^3$$

Applying this value to the overlay pavement having $a = 7.4$ and $h = 6$ in.

$b = \sqrt{1.5a^2 + h^2} - 0.675 \quad h = 7.06$ inches. For $K = 1113$, $b = 7.06$, l is 15 and $l/b = 2.13$.

$$P = \frac{S_i \times h^2}{0.3162\ (4 \log_{10} 2.13 + 1.0693)} = \frac{400 \times 36}{0.3162 \times 2.48} = 18{,}400 \text{ pounds.}$$

The safe load that the overlay can take is 18,400 pounds having a tyre pressure of 140 pounds per sq. in., whereas it was subjected to single wheel loads of 24,000 pounds having the same tyre pressure.

3.5. Incidence of Cracks in the Overlay

Although the method of determination of the supporting power of the overlay adopted in para 3.4 is approximate as there is no accurate method of determining the stress in an overlay slab in such a system, it is evident that considering the effect of temperature in addition to wheel loads, the overlay pavement was subjected to stresses more than it could safely bear. This fact is borne out by the evidence of cracks that had already appeared in the overlay slabs towards the ends of the runway. The runway having been extended at both ends with new concrete pavements, the overlay begins at 300 feet from the eastern end of the extended runway and stops at 450 feet from the western end. The full effect of the wheel load may not be brought to bear on the overlay except at the ends where the incidence of cracks was high. It was noteworthy, however, that the new slabs of 11 inches in thickness laid in panels of 5 feet square in the extended portions where the full effect of wheel loads was brought to bear, were standing up well without any incidence of cracking yet.

4. HAKIMPET RUNWAY

4.1. The Pavement

This concrete runway pavement was constructed in the year 1944 having an average thickness of 5 inches, laid directly on a moorum subbase without any metal base course. The pavement was constructed of ordinary concrete of 1 : 2 : 4 mix proportions. No record is available regarding the gradation of fine and coarse aggregates used or the strength properties. The aggregates used consisted of quartz ballasts containing over-sized metal exceeding $2\frac{1}{2}$ in. gauge. The quality of concrete was not high. The pavement was laid mostly in panels 30 feet long and 8 feet wide. The longitudinal joints were staggered as shown in Plate II.

Approximately thirteen per cent of the pavement slabs were damaged and replaced by new slabs upto 1956. The runway was subjected to single wheel loads not exceeding 6,500 pounds having tyre pressures of 80 pounds per sq. in. The damage to the slabs occurred not only at the ends of the runway but also at other stretches of the runway. Extensive damage to slabs occurred between 1,200 feet to 600 feet from the western end. The pattern of cracks and their incidence from the western end of the runway are shown in Plate II. Cracks also occurred in several slabs of the pavement from the eastern end. In this case, too, considerable damage to slabs occurred at a stretch approximately 2800 to 3,200 feet from the eastern end. The damage here is more than what occurred at the end of the runway. The extensive damage to slabs at stretches away from the runway ends was attributed to lower supporting power of the subgrade under such stretches in relation to the supporting power of subgrade at ends of runway. No investigation of the subgrade conditions was carried out due to lack of availability of requisite equipment.

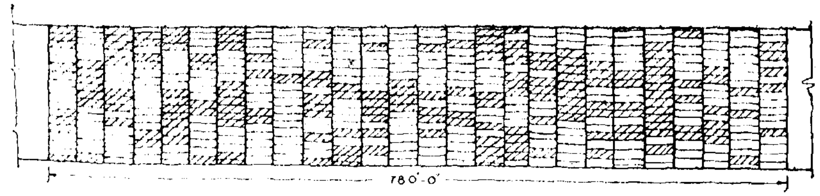
Due to extensive incidence of cracks in slabs all over the runway it was realised that the strength of the existing pavement was not adequate and it was necessary to strengthen it by an overlay.

4.2. Bituminous Overlay

Before construction of an overlay on the existing runway was taken in hand, an experimental bituminous overlay 780 feet in length was laid over a stretch of the runway over the entire width of 150 feet. This stretch starts from a point 2,700 ft from the western end and stops at 1,920 ft from this end.

Before the overlay was laid, all damaged slabs were replaced Plate II.

The stretch where the experimental overlay was laid had already shown considerable damage to slabs as is evident from Fig. 4. The subgrade condition, however, remained unchanged when this overlay was laid.



SHOWING THE POSITIONS OF BROKEN CONCRETE SLABS THAT WERE REPLACED BEFORE LAYING THE BITUMINIOUS OVERLAY

Fig. 4.

The overlay was laid out of the funds given for special repairs to the pavements. The thickness of the overlay was therefore, kept to such minimum thickness as would prevent the appearance in the overlay of the pattern of the expansion joints of the concrete pavements below the overlay. After several discussions, the following thickness of the overlay and the specifications of construction were adopted:

(a) Tack coat with Maxphalt 80/100 at the rate of 20 pounds } per 100 FS

(b) Base coat 2 in. thick consolidated with premix Shell Macadam of :

(i)	1 in. size metal	– 15 cu. ft.	per 100 FS
(ii)	3/4 in. -do-	– 5 cu. ft.	
(iii)	Fine river sand	– 1.2 cu. ft.	
(iv)	Mexphalt	– 70 lb	

(c) Top layer of 1 in. thick premix carpet as follows :

(i)	3/4 in. size metal	– 6 cu. ft.	per 100 FS
(ii)	1/2 in. -do-	– 3 cu. ft.	
(iii)	Fine river sand	– 4 cu. ft.	
(iv)	Mexphalt 80/100	– 72 lb	

(d) Top seal coat of :

(i) Fine river sand —2 cu. ft. } per 100 FS
(ii) Mexphalt 80/100—8 lb }

The bituminous overlay was completed in the middle of 1955 and was thereafter subjected to constant air traffic having maximum single wheel load as stated before. But it stood up well without showing any visible damage or crack for one year. By then the project for constructing an overlay on the entire runway was decided. As the airfield was designed for use by jet aircrafts and there was lack of indigenous tar and bitumen products that could stand up to the effect of jet exhaust and spillage of jet fuel, it was decided to strengthen the runway with a concrete overlay of 6 inches in thickness.

4.3. Concrete Overlay at Hakimpet

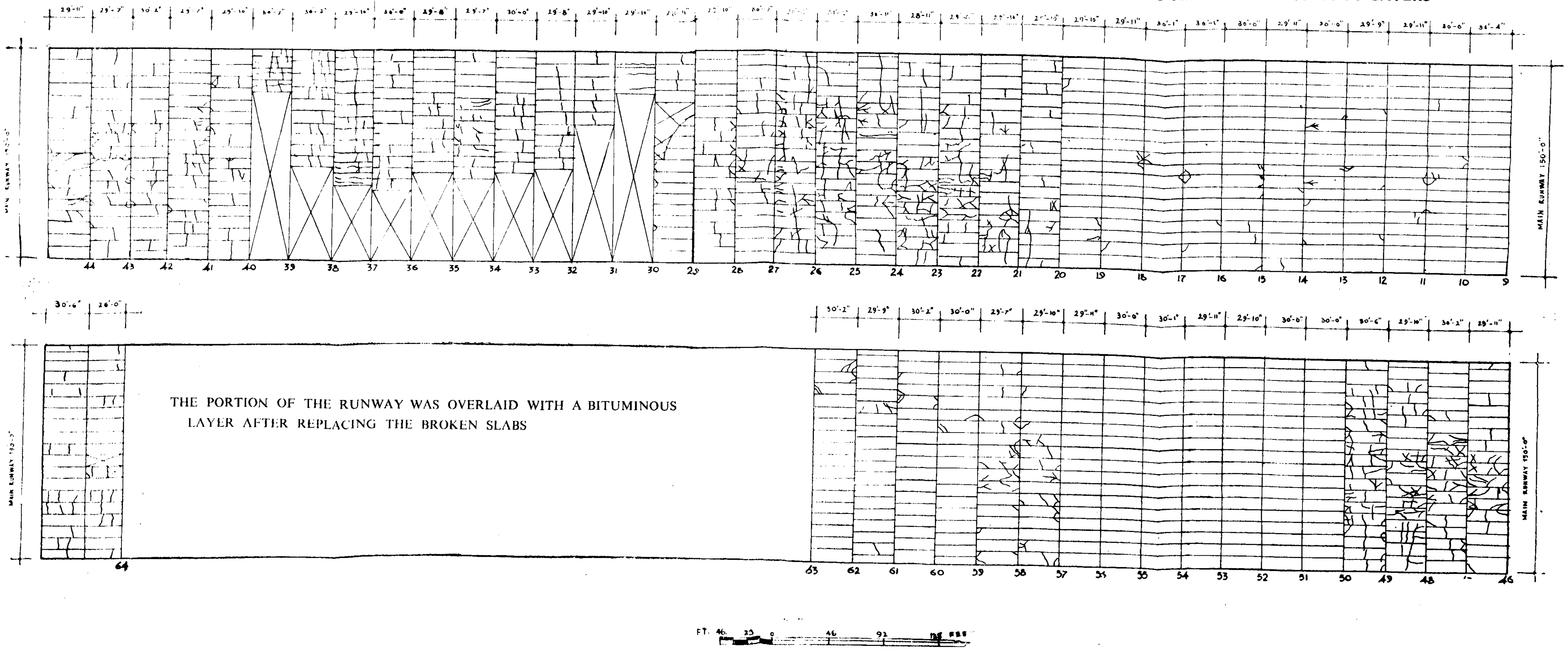
The construction of the concrete overlay started in the middle 1956. The specification of concrete accepted in the contract was the same as was used in the construction of the concrete overlay in Poona. The construction of the concrete overlay was carried out under strict laboratory control. The Hyderabad Engineering Research Laboratories investigated not only the design of the concrete mix but carried out extensive tests to determine strength and characteristics of field specimens. A detailed report on the design and execution of this concrete pavement is described in an article at pages 141 to 165 of the Annual Report of the Laboratories for 1956, (Hyderabad Engineering Research Laboratory Publication No. 10).

This overlay has stood up to the air traffic so far without any sign of failure of the pavement.

RESEARCH SECTION

PLATE II

SLABS MARKED X INDICATE THAT THE SLABS HAD SUNK AND USE WAS MADE OF BITUMINOUS LAYERS



DETAILS OF THE OLD RUNWAY PAVEMENT SLABS AT HAKIMPET SHOWING THE POSITIONS AND PATTERN OF CRACKS IN SLABS

"EXPERIMENTAL STUDY ON THE ADHESION OF BINDERS LIKE BITUMEN AND TAR NO. 3 WITH STABILIZED SOIL CRUST"

By

S. R. MEHRA, L. R. CHADDA & H. R. AGGARWAL*

[This Research Note was placed before the Research Organization Committee at Srinagar in June 1955. In accordance with the decision of the Committee, comments of the members of the Committee were obtained and sent to the Authors for replies. The comments of the members as well as the replies of the Authors given at pages 159–162 were placed before the Research Organization Committee at their meeting held at Hyderabad in January 1959 wherein it was decided that the note along with the comments of members and replies of Authors should be published in the Research Section of the Journal of the Indian Roads Congress.]

Bitumen and tar products are extensively employed in road construction for surface dressing purposes. The function of bitumen and tar in surface dressing is two-fold. Firstly, it binds together the particles of aggregate to give a hard wearing surface and secondly, it makes the surface waterproof and prevents ingress of moisture into the road crust. The performance of a binder for surface dressing depends largely on its adhesion with the aggregate.

In recent years, construction of stabilized soil roads as all-weather cheap roads has become fairly popular and is being widely adopted in places where suitable materials are easily available. Considerable lengths of surface treated stabilized soil roads have been constructed in the Punjab and are giving excellent service for the last 6 years under normal traffic conditions.

In fact, such black top roads can easily replace those constructed with water-bound specifications in regions of medium traffic at about half the cost and can give the same service.

Both bitumen (80/100) and tar No. 3 have been satisfactorily used for the surface treatment of stabilized soil roads. With a view to improve the surfacing of stabilized soil roads as well as to determine the various physical factors controlling their performance, an extensive experimental study was conducted in the laboratory under the following heads:

(1) Adhesion of 'bitumen' and 'tar' with a stabilized soil surface with and without priming.

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(2) Adhesion of binders with stabilized cement-soil mass.

(3) Effect of electrolytes on the adhesion of binders.

(4) Effect of moisture on the adhesion of binders.

ADHESION OF BINDERS WITH & WITHOUT PRIMING

Adhesion is a surface phenomenon and is entirely governed by a number of complex physico-chemical factors. For the determination of adhesion of these binders, laboratory experiments were conducted with soil blocks prepared by compacting a graded soil of P. I. 10.5 and sand content 42.0 per cent, mixed with 40 per cent brick aggregate one inch downwards, in iron moulds 4 in. dia. and $2\frac{1}{2}$ in. high to a density of 1.80-1.90. Soil aggregate blocks thus prepared were allowed to dry to constant weight. These compacted blocks represented specimen of stabilized soil road crust. Cement concrete blocks (1:2:4) of similar size but with a smaller height were prepared with their top surface embedded with small pieces of bajri. A number of small holes were kept in each concrete block so as to keep the interposed binder film exposed to the atmospheric conditions to the maximum possible extent to allow the evaporation of lighter oil present in the binder. The concrete blocks were used on one side to ensure cleavage on the other side, viz., the joint between the soil block and binder. In order to test the adhesive strength of different binders, the soil block and the concrete block (with gravel embedded in top surface) were joined together by a binder coated on the surface of each block. Each pair of blocks was kept in shade for a week and then tested for adhesion applying a direct pull by attaching a clamp to each block and pouring sand into a bucket hung from the lower clamp. In case of surface treatment with priming, the stabilized soil block was given a coat of shell primer No. 3 at 21 lb per 100 sq. ft. at 120° F. and allowed to penetrate the surface for a period of 24 to 48 hours required for complete penetration. Over this primer coated surface, the binder was applied as above and tested for adhesion in the same manner.

The following two binders were used in all the experiments :

Binder	Temperature at application
Tar No. 3	200°F. to 220°F.
Bitumen 80/100	350°F. to 375°F.

Although the above procedure of testing adhesion does not exactly simulate the kind of stresses that are likely to be induced in the crust, yet it gives an idea of the relative performance of different binders under identical conditions. The results of adhesion of different binders are given in Table 1.

TABLE 1

S. No.	Description of surface	Density	Adhesion of binders with & without priming. Adhesion in lb per sq. inch. With tar No. 3: Without priming coat	With tar No. 3: With Shell Primer No. 3 coat @ 21 lb/100 sq. ft.	With bitumen 80/100: Without priming coat	With bitumen 80/100: With Shell Primer No. 3 coat @ 21 lb/100 sq. ft.	Remarks
1.	Stabilized soil blocks with 40 per cent soft aggregate (loam)	1.8 to 1.9	6 to 8 lb	7 to 9 lb	15 to 18 lb	15 to 18 lb	Average of large number of tests (50-60)
2.	Loam soil compacted at optimum moisture without any aggregate	1.8 to 1.9	7 to 8 lb	8 to 10 lb	14 to 17 lb	15 to 18 lb	do.
	* (i) Sandy loam	1.8 approx.	6.7 lb	9.0 lb	11.7 lb	14.1 lb	Average of 4 tests
	(ii) Silty loam	do.	9.7 lb	9.3 lb	13.2 lb	13.9 lb	do.
	(iii) Silty clay loam	do.	9.1 lb	9.7 lb	13.7 lb	12.6 lb	do.
	(iv) Loam	do.	8.0 lb	10.3 lb	13.6 lb	15.2 lb	do.
3.	Burnt brick block	—	7.8 lb	—	22.2 lb	—	Average of six tests
4.	Concrete block with gravel embedded in top surface	—	10.8 lb	—	21.9 lb	—	do.

* Physical characteristics of the soils (i) to (iv) above are given in Table 2.

Soil blocks with and without soft aggregate of lesser density than 1.8 could not be tested properly as the blocks themselves gave way at much lesser loads than those required to break the adhesive bond.

TABLE 2

Physical characteristics of the soils

S. No.	Soil type	Liquid limit	P. I.	Sand per centage	Sulphate per centage	Optimum moisture
1.	Sandy loam	21.6	6.0	60.0	Nil	11.0
2.	Silty loam	33.3	10.3	15.5	Nil	13.0
3.	Silty clay loam	37.2	19.2	11.5	0.05	13.0
4.	Loam	25.9	10.9	48.0	Nil	11.5

It will appear from the above results that adhesion of bitumen (80/100) is fairly high as compared to tar No. 3, due mostly to its high viscosity. In selecting a suitable binder, however, there are other considerations also to be taken into account such as penetration, permanency of binder film in the presence of moisture, prevailing temperatures, etc. Moreover, priming the soil surface with shell primer No. 3 before application of the binder has no appreciable effect in increasing its adhesion. It was also noticed that smooth surface of the soil crust gave better adhesion than rough surface. It is further indicated in the above results that soil crust with 1.8 density gives the same adhesion with tar No. 3 as with brick.

The comparative adhesion of different binders also depends upon their viscosity; low viscosity no doubt is an aid to wetting the aggregate and soil surface but high viscosity gives better adhesion.

The study was extended to the surface dressing of cement-soil roads. Blocks of all the four soils commonly found in Punjab were made without any aggregate but with increasing percentages of cement, cured for a week and dried in the sun. The experiments were carried out under similar conditions as

described earlier. The physical characteristics of the soils selected for study are given in Table 2 and results of adhesion are embodied in Table 3.

It will appear from the data that adhesion of binders increases in all the four soils with increasing percentages of cement. Silty clay loam and loam soils give better adhesion than others with low percentages of cement. However, with 7.5 per cent or 10.0 per cent addition of cement, all the soils give almost equally good adhesion.

TABLE 3

Adhesion of binders with soil-cement blocks

S. No.	Soil type	Cement percentage	Adhesion in lb/sq. in. (Average of 4-5 tests)		Remarks
			With tar No. 3	With bitumen 80/100	
1.	Sandy loam	(i) 0	6.6	8.8	All the blocks separated from the binder layer on testing
		(ii) 2.5	7.7	12.5	
		(iii) 5.0	11.3	21.2	
		(iv) 7.5	12.8	22.1	
		(v) 10.0	13.3	25.3	
2.	Silty loam	0	9.1	11.2	
		2.5	9.1	14.4	
		5.0	13.3	18.7	
		7.5	12.3	24.6	
		10.0	12.1	25.3	
3.	Silty clay loam	0	7.3	17.3	
		2.5	9.1	18.7	
		5.0	10.6	29.7	
		7.5	10.2	29.0	
		10.0	10.2	27.5	
4.	Loam	0	9.0	14.1	
		2.5	10.3	16.5	
		5.0	15.1	17.3	
		7.5	16.5	24.6	
		10.0	16.5	30.5	

The blocks were compacted in Abbot's Apparatus giving 40 blows of the hammer weighing 5½ lb and falling through a height of 15 inches.

EFFECT OF TIME

Since the molecular orientation of viscous liquids proceeds slowly, the element of time may be considered to be an important factor in developing the maximum adhesion between an aggregate or compacted soil surface and the binder film. In view of this fact, it was considered desirable to test adhesion by giving longer periods of setting. The results obtained are given in Table 4.

TABLE 4

Showing the effect of time on the adhesion of binders

Period of contact with stabilized soil surface	Density	Adhesion in lb/square inch. With tar No. 3: With loam soil block without aggregate	With tar No. 3: With loam soil block with 40 per cent soft aggregate	With bitumen 80/100: With loam soil block without aggregate	With bitumen 80/100: With loam soil block with 40 per cent soft aggregate	Remarks
10 days ...	1.85 approx.	8.0	7.5	19.8	16.0	Average of 3 tests
1 month ...	,,	7.5	8.6	18.5	17.2	
2 months ...	,,	8.5	7.4	19.3	15.4	All the blocks separated from the binder layer
4 months ...	,,	8.5	8.4	19.0	16.8	

It will appear from the above Table that without traffic vibrations there is no appreciable improvement in the adhesion with longer setting periods.

EFFECT OF ELECTROLYTES

With a view to improve the adhesion of binders with a stabilized soil crust, addition of small concentrations of electrolytes in a soil mass was also tried. Loam soil blocks were

compacted with 1 per cent solution of common salt and 1 per cent lime slurry in water respectively. These were joined with concrete blocks using tar No. 3 and bitumen 80/100 and tested for adhesion after a period of one month. The results obtained are given in Table 5.

TABLE 5

Showing the effect of electrolytes on the adhesion of binders

Description of the soil block	Concentration of salt	Density	Adhesion in lb/sq. in. Tar No. 3	Adhesion in lb/sq. in. Bitumen 80/100	Remarks
(i) Loam soil compacted block	Nil	1.9	7.5	18.5	Average of 3 tests
(ii) Soil block with 1 per cent lime slurry	0.1 per cent on dry wt. of soil	1.85	7.5	16.7	
(iii) Soil block with 1 per cent common salt solution	do.	1.9	6.8	16.9	All blocks separated from binder layer on testing

The results given in this Table do not indicate any increase in the adhesion with this treatment. The experiments were repeated with higher concentration of these chemicals including calcium chloride as well, and were conducted under two different conditions. In the first case, the binder was applied on the surface of the soil block immediately after compaction, while in the second case it was applied after the block had been dried. Solutions of different chemicals with varying concentrations were taken at optimum moisture and added at the time of compaction.

The results obtained are given in Tables 6 and 7.

TABLE 6

Showing the effect of electrolytes (added during compaction) on the adhesion of binder. Binder used—Tar No. 3

S. No.	Description	Concentration of chemical added	Setting period of binder	Adhesion in lb per sq. in.	Remarks
	(i) Condition of application of binder—dry blocks				
1.	Loam soil block compacted with tap water	Nil	One month	8.2	Average of four tests
2.	Loam soil block compacted with 2.5 per cent calcium chloride solution	0.25 per cent on dry wt. of soil	do.	11.2	
3.	Loam soil block compacted with 2.5 per cent sodium chloride solution	do.	do.	10.0	
4.	Loam soil block compacted with 2.5 per cent lime slurry	do.	do.	10.4	All the blocks separated from binder layer
	(ii) Condition of application of binder—Binder applied just after compaction when blocks were retaining optimum moisture				
1.	Loam soil compacted block	Nil	One month	12.1	Blocks separated from binder layer
2.	Loam soil block compacted with 2.5 per cent calcium chloride solution	0.25 per cent on dry wt. of soil	do.	12.4	do.
3.	Loam soil block compacted with 2.5 per cent sodium chloride solution	do.	do.	12.0	do.
4.	Loam soil block compacted with 2.5 per cent lime slurry	do.	do.	16.5	Soil block broken

The blocks were compacted in Abbot's Apparatus giving 40 blows of the hammer weighing 5½ lb and falling through a height of 15 inches.

TABLE 7

Showing the effect of electrolytes (added during compaction) on the adhesion of binder. Binder used—Bitumen 80/100

S. No.	Description	Concentration of chemical	Setting period of binder	Adhesion in lb/sq. inch. (Average of 4 tests).	Remarks
	(i) Condition of application of binder—Dry blocks				
1.	Loam soil blocks compacted with tap water	Nil	One month	18.5	Blocks separated from the binder layer
2.	Loam soil blocks compacted with 2.5 per cent calcium chloride solution	0.25 per cent on dry wt. of soil	do.	22.4	do.
3.	Loam soil blocks compacted with 2.5 per cent sodium chloride solution	do.	do.	15.7	do.
4.	Loam soil blocks compacted with 2.5 per cent lime slurry	do.	do.	13.4	do.
	(ii) Condition of application of binder—Binder applied just after compaction when blocks were retaining optimum moisture				
1.	Loam soil blocks compacted with tap water.	Nil	One month	26.3	Blocks separated from the joint
2.	Loam soil blocks compacted with 2.5 per cent calcium chloride solution.	0.25 per cent on dry wt. of soil	do.	28.5	Soil block broken
3.	Loam soil blocks compacted with 2.5 per cent sodium chloride solution.	do.	do.	33.4	do.
4.	Loam soil blocks compacted with 2.5 per cent lime slurry	do.	do.	25.1	do.

The blocks were compacted in Abbot's Apparatus giving 40 blows of the hammer weighing 5½ lb and falling through a height of 15 inches.

It will appear from the above Tables that with Tar No. 3 there is some increase in the adhesion of binder film under both the conditions and it is just possible that under field conditions the effect may be more pronounced. With bitumen, however, there is an appreciable increase in adhesion only when applied on wet blocks: but under dry condition, the adhesion increases to some extent in the presence of calcium chloride only.

EFFECTS OF MOISTURE

Moisture plays an important role in soil stabilization and to attain maximum stability, compaction is carried out at optimum moisture. So, it is rather important to know to what extent the presence of moisture in the stabilized soil crust will affect the adhesion of the binder. The experiments were tried with a loam (P.I. 11.4 and sand content 40 per cent) compacted at optimum moisture and then dried to varying moisture contents. Adhesion was tested in the same way as described earlier. Results obtained are given in Table 8.

TABLE 8

Showing the effect of moisture (tap water) on the adhesion of binders

Description	Binder used	Moisture percentage retained from optimum quantity per cent	Adhesion in lb/sq. in.	Remark
Stabilized soil block (loam soil used)	Tar No. 3	Nil	6.9	Results given are average of 4 tests
		9-11	12.9	
		5-7	12.1	
Density 1.8 approx.	...	1-2	8.9	...
do.	Bitumen 80/100	Nil	13.7	All blocks separated from the binder layer
		11	21.5	
		8-9	20.0	
		6-7	18.5	
		3-5	16.4	

The results clearly indicate that presence of some moisture in the soil crust at the time of applying asphaltic binders substantially improves the adhesion of both types of binders. This fact should be of great significance in the surface treatment of stabilized soil roads.

On a field scale, the surfacing of stabilized soil roads is generally carried out after a period of a fortnight or so to allow

proper curing of the wearing course. Naturally, under these conditions, there may not be appreciable moisture in the surface to effect the adhesion of binders to a considerable extent. So, for practical application of this phenomenon under field conditions, it was considered desirable to continue experiments by sprinkling extra moisture on the surface of the soil crust before applying a binder coat. In these experiments, loam soil blocks compacted at optimum moisture to 1.8–1.9 density and then dried in the oven were tested. One side of the block up to ¾ in. was soaked in water for a short interval and then allowed

TABLE 9

Showing the effect of moisture (soaked afterwards by the dry surface) on the adhesion of binders

Description	Binder	Moisture percentage soaked afterwards	Adhesion in lb/sq. in.	Remarks
Stabilized soil blocks with 40 per cent soft aggregate	Tar No. 3	Dry	6.4	Average of 3 tests
		4.2	4.6	
		7.0	5.8	All the blocks separated from the binder layer on testing
Loam soil blocks without aggregate		Dry	8.4	
		4.3	4.8	
		6.5	4.6	
		8.9	5.0	
Burnt brick block		Dry	6.2	
do.		After soaking in water	5.6	
Stabilized soil blocks with 40 per cent soft aggregate	Bitumen 80/100	Dry	19.6	
		6.1	16.0	
		8.7	15.0	
Loam soil blocks without any aggregate		Dry	17.4	
		7.0	16.8	
		8.1	16.6	
Burnt brick blocks		Dry	27.0	
		After soaking in water	25.4	

to dry at room temperature to varying moisture contents. It was at this stage that a coat of binder was applied and later on tested for adhesion. The results obtained are given in Table 9.

These results reveal that addition of moisture to the surface of the soil crust, after letting it dry once, does not in any way increase the adhesion of the binder. This increase, as already

TABLE 10

Showing the effect of electrolytes (soaked afterwards in dry soil blocks) on the adhesion of binbers

Description	Electrolytes soaked & its concentration	Adhesion in lb/sq. in. With Tar No. 3	Adhesion in lb/sq. in. With Bitumen 80/100	Remarks
Stabilized soil blocks	Dry state	7.4	18.6	Average of 3-4 tests
Loam soil	Binder applied just after compaction at optimum moisture	13.0	29.0	
	Dry block soaked in tap water	3.6	13.6	All the blocks separated from the binder layer
	1 per cent calcium chloride solution soaked by dry blocks	3.8	14.9	
	2.5 per cent calcium chloride solution soaked by dry blocks	5.6	10.9	
	1 per cent sodium chloride solution soaked by dry blocks	5.6	10.3	
	2.5 per cent sodium chloride solution soaked by dry blocks	5.6	13.6	
	1 per cent lime slurry solution soaked by dry blocks	3.6	13.2	
	2.5 per cent lime slurry solution soaked by dry blocks	5.8	10.0	

TABLE 11

Showing the effect of moisture on the adhesion of binders with cement-soil blocks

Soil type	Moisture percentage	Blank soil blocks (i) Dry state	Blank soil blocks (ii) 8 per cent retained from * O. M.	Blank soil blocks (iii) 8 per cent soaked afterwards	2.5 per cent cement-soil blocks (i) Dry state	2.5 per cent cement-soil blocks (ii) 8 per cent retained from O. M.	2.5 per cent cement-soil blocks (iii) 8 per cent soaked afterwards	5 per cent cement-soil blocks (i) Dry state	5 per cent cement-soil blocks (ii) 8 per cent retained from O. M.	5 per cent cement-soil blocks (iii) 8 per cent soaked afterwards	7.5 per cent cement-soil blocks (i) Dry state	7.5 per cent cement-soil blocks (ii) 8 per cent retained from O. M.	7.5 per cent cement-soil blocks (iii) 8 per cent soaked afterwards	10 per cent cement-soil blocks (i) Dry state	10 per cent cement-soil blocks (ii) 8 per cent retained from O. M.	10 per cent cement-soil blocks (iii) 8 per cent soaked afterwards
Binder used—Bitumen 80/100																
1.	Sandy loam	8.8	10.7	Block broken	12.5	11.3	5.5	21.2	23.9	21.6	22.1	35.9	18.7	25.3	42.8	25.8
2.	Silty loam	11.2	21.7	12.1	14.4	13.6	15.8	18.7	21.7	23.1	24.6	24.6	25.3	25.3	31.2	29.0
3.	Silty clay loam	17.3	30.5	20.3	18.7	31.9	29.7	29.7	30.5	23.1	29.0	38.4	32.7	27.5	38.6	29.0
4.	Loam	14.1	20.2	12.9	16.5	18.7	18.0	17.3	29.7	23.1	24.6	34.0	30.5	30.5	37.7	34.1
Binder used—Tar No. 3																
1.	Sandy loam	6.6	7.8	6.5	7.7	13.2	9.1	11.3	12.0	11.6	12.8	14.7	9.9	13.3	13.9	11.2
2.	Silty loam	9.1	9.1	8.7	9.1	10.0	12.9	13.3	11.3	13.6	12.8	10.0	13.8	12.1	11.0	13.9
3.	Silty clay loam	7.3	11.1	9.5	9.1	11.3	11.6	10.6	13.4	11.3	10.2	11.3	14.3	10.2	12.0	13.7
4.	Loam	9.0	11.0	10.7	10.3	12.9	12.0	15.1	12.5	10.3	16.5	12.5	12.3	16.5	11.9	10.3

* O. M. means optimum moisture.

shown, could only occur when applied on freshly compacted soil mass. However, attempts were continued to find out if this could still be achieved by dipping the soil blocks in electrolyte solutions, thereby imparting an ionic activity on the surface of the soil crust. The results which are embodied in Table 10 show that even this treatment has no beneficial effect on the adhesion of binders.

The above study was extended to cement-soil compacted blocks. In these experiments cement-soil blocks were prepared at optimum moisture with different soils using varying concentrations of cement. Effect of moisture retained under different conditions on the adhesion of binders with cement-soil blocks was determined as discussed earlier. Physical characteristics of the soils selected for the study are already embodied in Table 2. The results obtained regarding adhesion of these binders with stabilized soil-cement surface are given in Table 11. It will be seen that the effect of moisture in increasing adhesion between cement-soil block and binder is well pronounced with bitumen 80/100 and moisture retained from optimum quantity is more effective than that introduced in later stages.

SUMMARY

It has been shown in this study that the adhesion of bitumen and tar film with a stabilized soil crust is controlled by a number of physical factors such as (i) density of road crust, (ii) viscosity of binders, (iii) moisture content of road crust, and (iv) nature of surface. By carefully controlling these factors, a substantial improvement in the adhesion can be achieved. Priming has not proved itself of much value in improving the adhesion. Addition of small concentrations of electrolytes also improves adhesion in the presence of some moisture.

The following conclusions have been drawn:

(a) For maximum adhesion with binder, the compacted soil in the crust should have a dry bulk density of not less than 1.8.

(b) Other considerations being equal, the more viscous binders seem to have better adhesion.

(c) Up to optimum moisture, the more the moisture in the crust the greater the adhesion. (It must, however, be borne in mind that once the surface is sealed with a bituminous binder, the crust will dry at an extremely low rate, and will, therefore, take a long time to develop stability. It may, therefore, be necessary to let the crust dry partly so as to attain stability and also enough adhesion with the carpet before surface treating it).

(d) In contrast to a rough surface, a smooth surface of the crust gives better adhesion. (The scrubbing of surface before surface treatment, as done in the water-bound macadam roads, should not, therefore, be done for stabilized soil roads, only superficial dust should be brushed off).

Comments of Members of the Research Organisation Committee

1. Comments by Chief Engineer, Punjab

The adhesion of a binder to a surface is essentially a surface phenomenon and depends largely upon the nature of the binder and the area of the surface in contact with the binder. In the experiments conducted in the Research Laboratory at Karnal, the adhesion of the binders to the stabilized soil was determined by ascertaining the force to pull apart the two specimens 4 in. dia. and 2½ in. high, one consisting of stabilized soil under test and the other made of a standard surface of concrete with gravel embedded in it. While the surface area of the soil block is constant, the area of the concrete block varies with the amount of gravel embedded in the concrete which is not a constant factor. This is likely to introduce certain discrepancy in the eventual results obtained.

Tars and bitumens are plastic materials which would flow under any stress howsoever small applied for a certain period of time. The rate of application of this stress is, therefore, an important factor in the work with bituminous material. From the description of the test given in the Paper, the rate of application of this stress has not been mentioned or taken into account. The ecological properties of binders are very important and are now being taken into account in the case of all works being done with bituminous binders. He is, therefore, of the opinion that the results tabulated in the Paper are not really comparable as this factor has not been taken into account in this study.

Further, the nature of the failure has not been indicated in any of the Tables. The failure could be of two kinds. It may be the separation of the binder from the soil surface or it may be the flow of the bitumen which measures viscosity of the binder rather than the adhesive force with which it is sticking to the soil surface. The fact that bitumen has given better adhesion with soil as compared to tar No. 3, as shown in the case of results given in Table 2, is an indication of the fact that the test used for the determination of the adhesion strength is not measuring this quality of the binders. Normally, one would have expected that the adhesion of tar to soil surfaces would be greater than that of bitumen on account of the higher viscosity of the latter and the lack of wetting properties. But the results given by the test are contrary to this theory.

Further, the temperature at which the tests are made is also an important factor and it is not mentioned in the report whether this was kept constant for all the tests.

It has been mentioned in the Paper that it has been observed that the application of primer does not affect the adhesion of the binder to the surface. This is also against what would be expected and further proves that the test does not really measure the adhesion of the binder to the soil surface but merely measures the viscosity of the binder in question.

It is, therefore, his opinion that the test evolved by the Karnal Laboratory does not, in fact, measure the adhesion of the binder to the stabilized soil surfaces and, as such, the results obtained are not indicative of the actual adhesive properties of the binders to the surfaces but merely measures the viscosity of the binders used. The results obtained are rather erratic which probably is due to the fact that the rate of application of the load is not constant and this produces the discrepancy in the results. He would, therefore, suggest that the study should be performed again after standardising the surface against which the adhesion of the binder is to be measured and, secondly, by applying the force to pull the test pieces apart at a constant rate. It is also essential that the temperature at which these tests are carried out should be kept constant.

2. Comments by Shri K. F. Antia

General: The findings of this Paper are very significant; if the results of tests are by any means a measure of relative performance of such construction in the field, then this information will readily help in establishing the best specification for the surfacing of *stabilized soils*. Possibly the interpretation could have been extended had some correlation been established between the results of these tests to actual performance of roads under identical conditions, since it is reported that such roads have been lasting satisfactorily for more than 6 years in the Punjab.

Conducting a shearing test of such surface courses instead of a pull test would possibly bring the results more near to the observed performance in the field, and possibly give a direct indication of the merits of various treatments. For this the specimen could possibly be shaped similar to the dimensions of the shear box with an intermediate block of concrete.

The results are also very revealing in regard to the use of reasonable percentage of cement, as cement content of over 5 per cent nearly doubles the adhesion in all the soils tested.

The number of tests and materials employed does not fully justify the inference that priming coat has not proved of value in improving the adhesion.

Conclusions 'c' and 'd' appear to be rather difficult to be obtained in practice in that if the moisture content of 6 per cent is to be retained for improving the adhesion of the binder, under such condition the procedure for removing superficial dust by brushing is not likely to be possible; moreover, without very experienced staff it would be difficult to judge the moisture content to be retained in the crust.

It would have also been more instructive had the traffic conditions under which this type of surface dressings had been successful, been mentioned.

With regard to the reference to satisfactory performance of stabilized soil roads for over 6 years in the Punjab, an indication whether such pavements with a bitumen or tar surfacing have satisfactorily withstood the punishment of bullock-cart traffic would be of considerable value. Details of composition of such pavements would also be valuable.

3. Comments by Shri K. K. Nambiar

The research deals with laboratory tests conducted on the adhesion of bitumen and tar binders on stabilized soil base.

It is noticed that the conclusions, viz., a, b, c and d arrived at are solely on the results of laboratory tests on stabilized samples. It is necessary to correlate the laboratory results with those obtained in the field before the conclusions can be confirmed.

4. Comments by Shri M. S. Bisht

It is considered that a few experimental stretches or roads may be laid to observe the behaviour of stabilized surface treated with binder in the presence of moisture to confirm the laboratory results.

Replies of the Authors to the Comments of Members.

1. Reply to the comments of the Chief Engineer, Punjab

Surface area of the concrete block does not come into picture, as adhesion was measured with the stabilized soil surface and not with the concrete surface.

The rate of application of stress was kept uniform in all the tests by regulating the flow of sand in the lower bucket. It was of the order of 40 lb per minute.

The nature of failure in all the tests was the separation of the binder layer from the stabilized soil block. This type of failure truly represented the relative adhesion of different soil surfaces with binder.

The fact that bitumen has given higher test value than tar No. 3 is due to its greater viscosity, resulting in better adhesion.

The blocks were tested at the room temperature prevailing, the daily variations which were of the order of $\pm$ 2°C. were too low to affect any change in the viscosity of the binder.

It is true that results may be different at temperature varying much above or below the temperature of tests.

Regarding primer treatment, reduction in adhesion caused by priming may be due to loss of cohesion of the soil particles on the top surface. Similar results have been obtained in the field.

The method of testing adopted was a measure for adhesion and not viscosity. This is the very method which has been suggested by the Chief Engineer, Punjab, himself in the last para of his comments.

2. Reply to the comments of Shri K. F. Antia

These observations have been recorded in the past but not in detail. Detailed observations are, however, being collected to correlate them with laboratory results.

The object of the laboratory investigation was to get relative adhesive properties of tar No. 3 and bitumen with a stabilised soil crust under identical conditions. It is agreed that a shear test will give further information.

As a result of subsequent work in this laboratory, it has been observed that adhesion with primer depends upon the type of solvent used. In the laboratory tests primer prepared with diesel oil was tried which generally gives low adhesion. Further field work is required on this aspect of the problem.

The field laboratory staff attached to the construction of stabilized soil road can easily determine the moisture content in the crust.

" BITUMEN IN ROAD SURFACING— DEVELOPMENT OF COLD FINE ASPHALT IN MADRAS "

By

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CONTENTS

SYNOPSIS

Laboratory investigations were taken in hand during 1956/57 to assess the suitability of utilising a large quantity of crusher run granite fines available at Madras Corporation's quarries and crushing plant at Pallavaram. Initial tests revealed that this aggregate would be suitable for preparing cold fine asphalt which is used extensively in Western countries for road surfacing works. This Paper gives details of laboratory and field trials carried out along with their results and of subsequent developments towards the large-scale use of this material on Madras City roads.

1. INTRODUCTION

Amongst the many specifications for bituminous road construction, ranging from light premix chipping carpets to densely graded hotmix asphaltic concrete pavings, is cold fine asphalt, which during the last two decades or so has been extensively used for road surfacing work both in the U. K. and countries in Continental Europe. Hundreds of

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The Paper does not deal with the surface dressing of stabilized soil roads.

Surfaced stabilized soil roads in the Punjab have withstood punishment of mixed traffic comprising bullock-carts, lorries, etc. Surfacing has been done using bitumen 80/100 as well as tar No 3.

3. Reply to the comments of Shri K. K. Nambiar

The suggestion of Shri K. K. Nambiar was noted. It is obvious that field trials have to be conducted before any applicable results can be arrived at.

4. Reply to the comments of Shri M. S. Bisht

Shri Bisht's idea was very much appreciated by the Authors and they agreed that it would be very helpful to try such field experiments.

Information Section

1. BITUMEN IN ROAD SURFACING — DEVELOPMENT OF COLD FINE ASPHALT IN MADRAS

2. THE INVESTIGATION, DESIGN AND CONSTRUCTION OF SUBMERSIBLE BRIDGES

3. RECOMMENDED PRACTICE FOR NUMBERING BRIDGES AND CULVERTS

miles of roads in these countries have been surfaced very successfully by using this material.

It has been used in the form of thin carpets ($\frac{3}{4}$ in. to one inch thickness) for resurfacing existing bituminous roads, on binder courses of bitumen macadam and for covering stone setts. It has been extensively used for surfacing footpaths, railway platforms, playgrounds and for general patching works. It also forms a useful adjunct to some of the newly constructed heavy duty roads by providing $\frac{3}{4}$ in. thick temporary surfacing for carrying construction traffic, etc., before the final asphaltic concrete wearing course is laid. In itself, it provides a non-skid wearing surface, but for improved appearance, the general practice is to roll in pre-coated $\frac{1}{2}$ in. size stone chips after laying cold fine asphalt.

2. GENERAL DESCRIPTION

Consisting of a fine crushed aggregate mixed with bitumen, the principle of this type of mix is the mechanical stability imparted to it by the interlocking of aggregate particles under compaction. Though it is generally laid warm as received direct from the mixing plant, it can be, due to its relatively low binder content and low binder viscosity, so designed as to remain workable for a long period and then laid cold. The word 'cold' may appear to be a misnomer as heat is always used while mixing but is quite appropriate considering the possibility of its eventual use in cold state.

Mixing and laying methods have been incorporated in the British Standard Specification 1690 : 1950, but here it should suffice to mention the following specifications for the ingredients according to this standard :

(a) **Aggregate :** Clean, hard, durable, crushed igneous rock or limestones of the grading given below :

Passing $\frac{1}{8}$ inch	BS Sieve	...	100 per cent	by weight
,, No. 7	,,	...	75-100 per cent	,,
,, No. 25	,,	...	35-60 per cent	,,
,, No. 100	,,	...	15-30 per cent	,,
,, No. 200	,,	...	5-15 per cent	,,

Generally, most $\frac{1}{8}$ inch down quarry fines will meet these limits except possibly the fraction passing 200 mesh which can be made good by the addition of a mineral filler.

(b) **Bitumen :**

Specific gravity at 60 degrees Fahrenheit	...	0.95–1.10
Penetration at 77 degrees Fahrenheit	...	180 min.
Viscosity at 104 degrees Fahrenheit using standard tar viscometer	...	20 min.

(c) **Mix Composition :**

Type of aggregate	Soluble bitumen Percentage by weight of mixed material	
	Min.	Max.
Igneous rock	4.5	6.0
Limestone	4.5	6.0
Slag	4.5	7.5

3. PRELIMINARY WORK IN MADRAS

3. 1. Materials Used

(a) **Aggregate :** As mentioned earlier, mechanical stability given by the interlocking of the aggregate particles under compaction being the key to the success of this mix, only a crushed aggregate is suitable. Under traffic, soft aggregates are likely to get crushed resulting in higher fines content. Their use leads to a smooth slippery surface which suffers from fretting.

In Madras, crushed quarry fines, which is not exactly a waste material, are available in large quantities at the Madras Corporation's crushing plant at Pallavaram. These quarries yield granite of a very hard and durable variety which has been extensively used for road works in and around Madras. A sieve analysis conducted on a representative sample from Pallavaram gave the following results:

Size of sieve			Percentage of total weight passing
3/16 in. BS sieve		...	100
No. 7	,,	...	91
No. 14	,,	...	73
No. 25	,,	...	55
No. 36	,,	...	46
No, 52	,,	...	36

Size of sieve		Percentage of total weight passing
No. 72 in BS sieve.	...	28
No. 100 ,,	...	22
No. 200 ,,	...	9

Fig. 1 shows that this aggregate falls well within the grading limits specified by BSS 1690: 1950. Conforming not only to the particle size limits, Pallavaram granite particles are also ocubical in shape with a rough surface texture which greatly helps in obtaining a sand paper textured non-skid riding surface.

(b) **Binder**: In addition to holding the aggregate particles together, the role of binder in cold fine asphalt where stock piling is required is to retain the mix in a workable condition. Therefore, the type of binder may range from a soft straight bitumen to a medium viscosity cutback, the actual choice being governed by the nature of aggregate, mixing facilities and working conditions. While low viscosity is required when the mix is being laid, the binder should exhibit substantial increase in viscosity with increasing traffic compaction to prevent possible deformations under traffic.

It was revealed by initial experiments conducted in Madras that 300 to 400 penetration bitumen was quite suitable for the aggregate available; the general practice in European countries is to use 400-500 penetration bitumen. Where stockpiling is not required, straight 180/200 penetration bitumen can be used which should give higher initial stability value on occasions where cold fine asphalt can be mixed and laid within a matter of a few hours.

Owing to non-availability of such soft grades in India, trial blends were made in the Burmah-Shell Laboratory to evolve a formulation suitable for the Pallavaram aggregate. Straight grades of bitumen Mexphalte 80/100 and Spramex 180/200 were blended with varying percentages of fluxes (light diesel oil and furnace oil were selected because of their low volatility) for trial purposes. Given below are their compositions and penetration respectively:

No.	Bitumen grade	Flux	Flux percentage by weight of bitumen	Penetration at 25 degrees Centigrade
1.	Mexphalte 80/100	Light diesel oil	5	225
2.	,,	,,	7	326
3.	,,	,,	8	415
4.	Spramex 180/200	,,	5	466
5.	,,	,,	6	480
6.	,,	Furnace oil	5	340

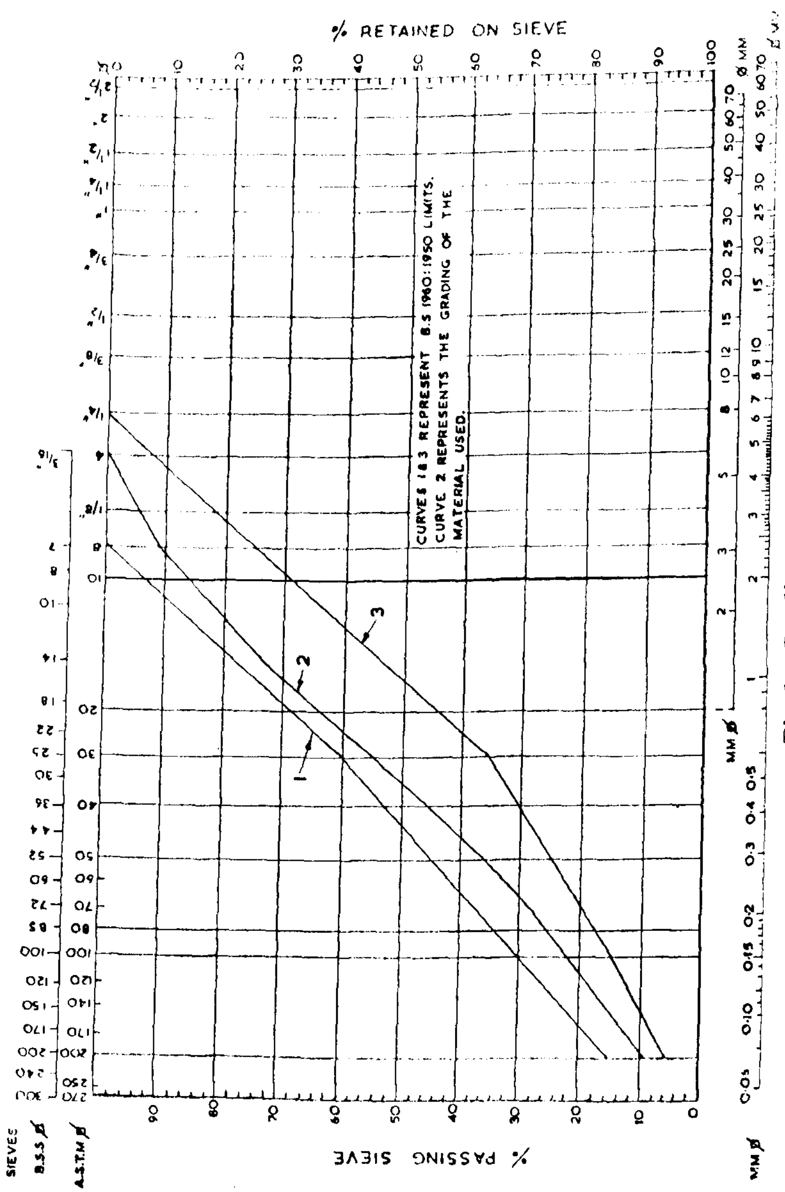

Fig. 1. Grading curve

A satisfactory mix should have a 'soft sugar' feel as against a hard, gritty and dry feel. A quick test is to squeeze some of the freshly manufactured and cooled mix in the hand; it should compress into a ball and retain its shape but should also readily crumble again when pressed between fingers and the thumb. While squeezing, however, very little of the mix should adhere to the hand. The behaviour of the mix in stockpile was assessed by studying the workability of small quantities of the mix in the laboratory. The selection of the binder was made on the texture and consistency of the mix when it cooled and its behaviour in the stockpile.

On this basis, after preliminary trials, blends with composition as detailed in items (2) and (6) of the Table on page 168 were selected and used in the experiments. Trials conducted with blend No. 1 proved unsatisfactory from the point of view of stockpiling.

It is to be pointed out that a wide range of binders can be used in cold fine asphalt, but field trials in Madras were limited to only two of the above binders. Furthermore, the percentage of flux and the type of flux used in these experiments cannot be taken as a standard; it may be necessary to try variations depending upon the aggregate available and prevailing working conditions.

(c) **Binder content** is a function of grading and nature of aggregate. Besides imparting mechanical stability to the mix, a correctly graded aggregate will ensure a minimum binder content compatible with performance. Excess of fines (passing 200 mesh) makes coating difficult whereas a preponderance of coarser particles gives a 'gritty' mix which is difficult to compact. The optimum binder content for cold fine asphalt is that which is just sufficient to hold the mixture together during the initial period after laying, before compaction by traffic knits it firmly. The amount should neither be too little nor too much so as to ensure the properties of the mix for storing, cold laying and durability. To some extent binder content depends on traffic conditions. Where ultimate compaction is expected to be slow due to light traffic conditions, it might be advisable to slightly increase the binder content. In the laboratory trials, mixes were prepared with 4.5, 5, 5.5 and 6 per cent of binder by weight of aggregate.

3.2. Laboratory Trials for the Preparation of the Mix

1500-gram batches were prepared. Both aggregate and binder were heated to 120 degrees Centigrade and the mixing carried out by hand; the mixing temperature not exceeding 150

Photo 1.

Bus stop on Whannels Road laid on water-bound macadam in February 1957

Photo 2.

Mount Road after 2 years of service; there is evidence of recurrence of cracks on the original surface due to wearing of the ½in. thick carpet laid in March 1957

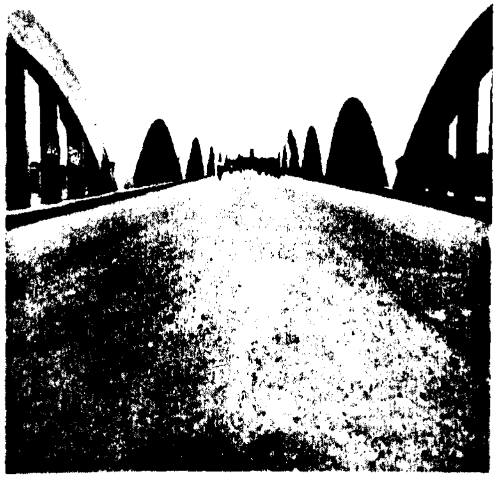

Photo 3.
Napier Bridge, Madras. One-third width of the road on either side surfaced in April 1957

Photo 4.
Egmore High Road surfaced in July 1958

Photo 5.
Compaction of laboratory sample of cold fine asphalt by wooden hand-roller

degrees Centigrade. This temperature should be as low as is compatible with satisfactory coating of the aggregate. Overheating of either the aggregate or binder would obviously drive off the volatile fractions of the flux, resulting in a mixture having poor storage properties. It was difficult to obtain a satisfactory mix by hand below 100 degrees Centigrade. The time taken for preparing each batch of the mix was about 5 to 7 minutes.

The mixes prepared with the above mentioned cutback bitumen were kept in storage for 7, 14, 21 and 60 days. They were then placed in wooden trays of 12 in. × 6 in. × ½ in. size and rolled with a small wooden hand roller weighing 450 grams, with the maximum pressure which the operator could apply. The mixes thus rolled were kept in the open sun and every half hour, between 10 A.M. and 5 P.M., 20 passes of the roller were given. This procedure was adopted to simulate the field conditions as far as possible where the carpet is supposed to be consolidated completely by pneumatic tyred traffic. Under this compaction, the surface became sufficiently hard to withstand maximum thumb pressure successfully. It took about 38 to 72 hours for the surface to become hard.

The results of these trials, as detailed in Annexure 1, may be summed up as follows :

I. By using 4.5 per cent binder content it was found that, for the aggregate used, the mix was difficult to consolidate even after extra rolling.

II. Other factors being equal, the rate of setting is quickened as :

(a) the binder content is increased within the limits normally specified, and

(b) dusting the surface with small percentage of limestone dust or cement.

III. The full inherent strength of the cold fine asphalt carpet is obtained only by the traffic that follows the initial rolling. Mere age does not seem to increase the strength.

IV. Even after 2 months storage, the mixes are easily workable.

As cold fine asphalt does not possess any inherent stability until laid on the road, any attempt to determine the strength of a test specimen prepared from freshly made material will give only misleading results and, as such, no tests were carried out. On the road it is expected that a cold fine asphalt surfacing should gradually wear in thickness without disintegrating. A hardened crust is formed on the top which takes the wear and tear of traffic with the material underneath remaining soft for a long time. It may normally take an appreciable time under average traffic conditions for the surface to become fully consolidated and to obtain its resistance to deformation.

3.3. (a) **Initial field trials**: As results of these trials conducted in Burmah-Shell Laboratory were encouraging, field trials were carried out by Madras Corporation authorities with the assistance of Burmah-Shell Bitumen Department. A Millars hotmix plant equipped with drying, screening and mixing unit at the Corporation Central Asphalt Plant was used for the manufacture of the mix.

As already mentioned, it was possible to get aggregate of a suitable grading from Pallavaram quarry for immediate use, a screen being used to reject any oversize particles. The mixing temperature was kept as low as possible (about 150 degrees Centigrade) to avoid driving off the volatile fractions of the flux.

The difficultly encountered while mixing was that the cyclone dust collector attached to the hotmix plant removed the finer particles of the aggregate as well, the resulting aggregate fed to the mixing plant being very coarse without any filler. As this dust collector induces the draft required for the burner, it was not possible to shut it off. The trouble was overcome by collecting the fines from the cyclone collector and adding them in the required proportion to each batch of the mix. Mixes were produced with 5 and 6 per cent binder content. The binder used was a cutback prepared with 80/100 penetration bitumen and light diesel oil. It was heated to 115 degrees Centigrade and added to the dried aggregate. It took about 2 minutes for complete coating of a batch of 800 lb. The mix was then discharged.

The mixed material was stockpiled on a clean platform and covered with a tarpaulin to avoid contamination by dust and water. Mixes prepared with 5, 5.5 and 6 per cent binder content stockpiled for 25 and 30 days were found to be easily workable. In fact, a small quantity of mix left over with a 5.5 per cent binder content was easily workable even after 45 days. Therefore, as the Corporation felt that their storage period was unlikely to exceed 45 days, further trials were not carried out.

(b) **Laying**: Experimental stretches were laid on the approach road to the Central Asphalt Plant, Egmore, where the intensity of traffic is about 750 tons per day, predominantly heavily loaded lorries and tractors. This portion of the road

where cold fine asphalt has been tried is also subjected to sharp turning, deceleration and acceleration. On the existing black top surface, 3/4 inch thick cold fine asphalt was spread as a renewal coat. Depressions in the surface were filled with precoated chips and allowed to be consolidated thoroughly by traffic before laying the cold fine asphalt. The filling of depressions is important since the renewal coat of the cold fine asphalt should be of uniform thickness. If the thickness varies from place to place, the compaction will be unequal and as such any unevenness in the old surface will show through. This was demonstrated by laying a short stretch of cold fine asphalt at a place without first filling the depressions; the surface looked true to level in the beginning, but subsequently developed corrugations.

After filling up the depressions and allowing them to consolidate under traffic for a few days, the surface was cleaned free of dust, dirt, etc., and a tack coat was applied. The quantity of bitumen to be used in the tack coat should be varied with the condition of the surface and be just sufficient to give as thin and even an application as possible. It is difficult to obtain a very thin film using neat bitumen, unless it is applied with a mechanical sprayer and as such a bituminous emulsion can be used very effectively. Colas was used in the tack coat at the rate of about 10 lb per 100 sq. ft. For purposes of comparison straight run bitumen of 80/100 penetration was also used but it was not possible to apply less than 15 lb per 100 sq. ft. evenly by hand pouring methods.

On delivery at site, the mixed material was placed in heaps on a clean, hard surface. The cold fine asphalt was spread by shovels and raked to a loose uniform layer of approximately twice the compacted thickness required. Handling, spreading and raking of this material was very easy when compared with other types of bituminous mixes.

Compaction of the cold fine asphalt was carried out with an 8-ton roller soon after spreading. The usual precautions in rolling as for any other bituminous carpet, i.e., moistening the roller wheels and keeping them free from any deleterious matter were taken. After an interval of 24 hours, the surface was rolled again. On surfaces that are not subject to intensive pneumatic tyred traffic, care must be taken to roll the carpet thoroughly. The road was opened to traffic immediately after final compaction. In the initial stages, traffic markings and in some cases hair cracks were noticed on the surface, but the ironing action of continuous traffic proved them to be of a self-healing type.

Another trial stretch of 1/2 inch thickness was laid on Mount Road which is one of the busiest thoroughfares in Madras. This stretch laid in 1957 is to-date performing very satisfactorily.

Short stretches of cold fine asphalt surfacing were also provided on :

(a) a water-bound macadam base on Whannels Road near a bus stop to a thickness of 3/4 inch.

(b) a footpath on Casa Major Road to a thickness of 1/2 inch using a richer mix with 6 per cent binder content to obtain better initial compaction. A vibratory roller with compaction equivalent to a 2-ton roller was used.

In all the above trial stretches the thickness of carpet laid was 1/2 inch or 3/4 inch only. It is, however, desirable that carpets of 3/4 inch to 1 inch thickness should be laid in preference to 1/2 inch thickness.

In addition to these road works, some trial patches were also carried out for which purpose cold fine asphalt proved ideal. After removing all loose material, the edges of the pot-holes were cut vertically and a paint coat of bitumen given. The mix was put in and consolidated by hand rammer and traffic allowed. This quick and efficient patching method was found very satisfactory and the Corporation authorities commenced sending the material packed in gunny bags to the work site. For pot-holes deeper than 3/4 inch it is advisable to use premix chippings to obtain a uniform level before applying cold fine asphalt.

4. CONCLUSION

These laboratory and field trials revealed that mixes prepared with 5.5 per cent binder content (the binder being a blend of mexphalte 80/100 fluxed with 7 per cent light diesel oil by weight of bitumen) gave better results both from the point of view of stockpiling and actual field performance. Hence, for the extensive work as outlined in the following paragraph, the Madras Corporation adopted this specification. The total cost of materials, mixing and laying a 3/4 inch thick carpet of cold fine asphalt in Madras City worked out to Rs 26.38 per 100 sq. ft. A detailed break-down is given in Annexure 2.

The road trials done in March 1957 proved very satisfactory and after observing their behaviour for about a year, the Corporation authorities adopted this specification on a fairly

large scale. Till about the middle of 1959, about 17 lakh sq. ft. of road surface (equivalent to 30 miles length of road 12 ft wide) in Madras City have been covered with cold fine asphalt, among which the following stretches are worth mentioning:

Mount Road	...	16 ft × 800 ft
Egmore High Road	...	20 ,, × 1320 ,,
Napier Bridge	...	30 ,, × 900 ,,
Purasawalkam High Road	...	20 ,, × 300 ,,
Poonamalle High Road	...	30 ,, × 500 ,,
Gandhi/Irwin Road	...	40 ,, × 500 ,,
Sir Muthuswamy Iyer Road (Dual carriageway)	...	2 × 12 feet × 1000 feet

ACKNOWLEDGMENT

D. R. Kohli & S. M. Mohideen are grateful to the General Manager, Burmah-Shell Oil Storage & Distributing Co. of India Ltd., Bombay, for permitting publication of this Paper.

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"THE INVESTIGATION, DESIGN AND CONSTRUCTION OF SUBMERSIBLE BRIDGES"

By

D. JOHNSON VICTOR*, B.E., A.M. Inst. H.E.

CONTENTS

SYNOPSIS

This Paper discusses the various points to be considered in the design of a submersible bridge. Some of the problems encountered are peculiar to submersible bridges and they need careful attention of the investigating and designing engineers. The value of engineering judgment rather than mere application of standard formulae for calculating flood discharge is stressed. Some of the aspects of design which need further experimental research are mentioned. The particulars that are to be carefully gathered and noted by the engineer investigating a submersible bridge project are listed in the Appendix.

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19

Annexure—1

S. No.	Percentage of binder	No. of days kept in storage before laying	Rolling	Special treatment, if any	Time taken for the surface to become hard - in hours: 180/200+5 per cent F.O.	Time taken for the surface to become hard - in hours: 80/100+7 per cent LDO	Remarks, if any.
1.	4.5	7	*	Nil	...	...	It was found from the behaviour and appearance of the mix that the binder content is rather on the low side and the mix difficult to consolidate. After the same amount of rolling for about 3 days, the surface did not become as hard as with mixes having higher percentage of binder. It also disintegrated very easily.
2.	5.0	7	*	Nil	72 hrs.	60 hrs.	
3.	5.0	14	*	Nil	66 ,,	56 ,,	
4.	5.0	21	*	Nil	60 ,,	50 ,,	
5.	5.5	14	*	Nil	60 ,,	50 ,,	
6.	5.5	60	*	Nil	54 ,,	48 ,,	
7.	6.0	14	*	Nil	56 ,,	48 ,,	
8.	6.0	60	*	Nil	46 ,,	38 ,,	
9.	5.5	14	*	Dusting the surface with small percentage of cement/lime.	48 ,,	38 ,,	Dusting the surface with small percentage of cement or lime seems to quicken the set of the surface.
10.	5.5	14	Only initial rolling. No subsequent rolling.	Nil	The surface did not become hard.		Ageing does not seem to improve the strength of the carpet. The strength is obtained only by the consolidation of the carpet by traffic.

* Rolling : Initial rolling+15 × 20 passes of the roller every day at an interval of half an hour between 10 A.M. and 5 P.M.

Annexure—2

I. Cost of manufacture of 3 cu. ft. of confined volume of cold fine asphalt using Madras Corporation's Millars Hotmix plant:

	Rs nP
4 cu. ft. (loose) quarry fines at Rs 30 per 100 cu. ft.	1-20
Weight of binder required at 5.5 per cent is 27 lb including wastage (weight of quarry fines per cu. ft. is 120 lb)	
Cost of 25 lb of Mexphalte 80/100 at Rs 310 per ton	3-37
Cost of 2 lb of light diesel oil at Rs 1/50 per gallon	0-38
Labour for mixing plant	1-25
Running charges of the plant	2-75
Sundries	0-55
	9-50

II. Cost of laying 3/4 in. thick cold fine asphalt for 100 sq. ft. $6\frac{1}{4}$ cu. ft.

of mix at Rs 9-50 per 3 cu. ft.	19-75
10 lb of Mexphalte 80/100 for tack coat	1-34
Labour for 100 sq. ft.	3-12
Rolling	1-69
Sundries	0-48
	26-38

1. INTRODUCTION

The subject of submersible bridges has not received its due share of attention from engineers all over the world and literature available about their design and performance is scarce. Submersible bridges are not commonly adopted in Western countries and hence in standard text books on bridges we often find that the authors pass off with just the definition of the term. It is rather disapppointing to note that this has been repeated to some extent in some of the standard Indian manuals as well. Probably it is partly due to the fact that these bridges appear less impressive and have been generally adopted on minor roads for linear waterways of smaller magnitude. But it is not often realised that within its limitations the submersible bridge offers a cheaper means of crossing than a conventional high level bridge, the cost of which may sometimes be such as to eclipse all chances of providing a crossing at all. Thus in under-developed countries like India, where finances for road works are not quite abundant, the potentialities of submersible bridges are great indeed. It is gratifying to see that the Highway Subcommittee of ECAFE have recently recommended[1] the adoption of submersible bridges on low cost roads.

2. DEFINITIONS

It would be worthwhile to note the definitions of the following terms which are used in this Paper.

2.1. Submersible Bridge

A submersible bridge is a structure designed to be overtopped during heavy floods, but having its road formation level fixed in such a way that the interruption caused to traffic during floods will not exceed three days at a time and six times in a year.

2.2. Extraordinary Flood Level (E.F.L.)

The extraordinary flood level is the level of the highest flood that has been known to have ever occurred at the crossing or the calculated level of the highest flood.

2.3. Maximum Flood Level (M.F.L.)

Maximum flood level is the level of the highest flood that may occur quite frequently, say, once in ten years or so.

.4. Ordinary Flood Level (O.F.L.)

ırdinary Flood Level is the level of a high flood which is ted normally to occur every year.

.5. Low Water Level (L.W.L.)

ıow water level is the level of the surface of the stream ally obtaining in dry weather.

3. CHOICE OF A SUBMERSIBLE BRIDGE

'he submersible bridge is essentially a low-cost bridge and :ompromise between a high-level bridge and a bed-level way or paved dip. It is best suited when the difference en the ordinary flood level and the maximum flood level or ordinary flood level is considerable and when the site is :t to wide fluctuations of floods of short duration. Also type of bridge can be adopted only on roads of minor 'tance and less traffic, where interruption to traffic for short ions of upto three days at a time for a maximum period of :en days in a year will not be of much consequence. Thus ıd that submersible bridges are generally most suitable for in hilly terrain where the streams are near to their origin, ıinor district roads and village roads and for roads sponby community projects where low cost is of great import-
Where the cost of a high level bridge at a particular site Major District Road of lower traffic intensity, say, less than tons per day, be such as to delay indefinitely the provision crossing at the site and where a submersible bridge at a cost could serve well enough, then, as a special case, ion of a submersible bridge on a Major District Road may ıe recommended. Such a case as this, though less frequent, still be met with in many less developed States of our ıry. But on State Highways and National Highways ersible bridges should not be adopted irrespective of cost. cide whether a submersible bridge would be preferable to a level bridge at a particular site where it is admissible, we to study the flood level chart and the bank conditions. ıuld be easily seen from Figs. 1 and 2 that at a site having section as in Fig. 1 and flood levels as in Fig. 2, a ersible bridge could serve well enough if the intensity of : is not quite high. The fixation of the road level is a very rtant point in the design of submersible bridges. This is ssed in greater detail later. From Fig. 1 the difference in between a submersible bridge and a conventional high level ;e at this site should be quite evident, as a high level bridge

would have its bottom of slab at 83.00, i.e., 20 feet above that for a submersible bridge.

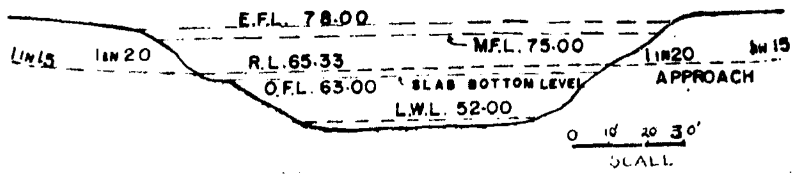


Fig. 1. Cross section at site

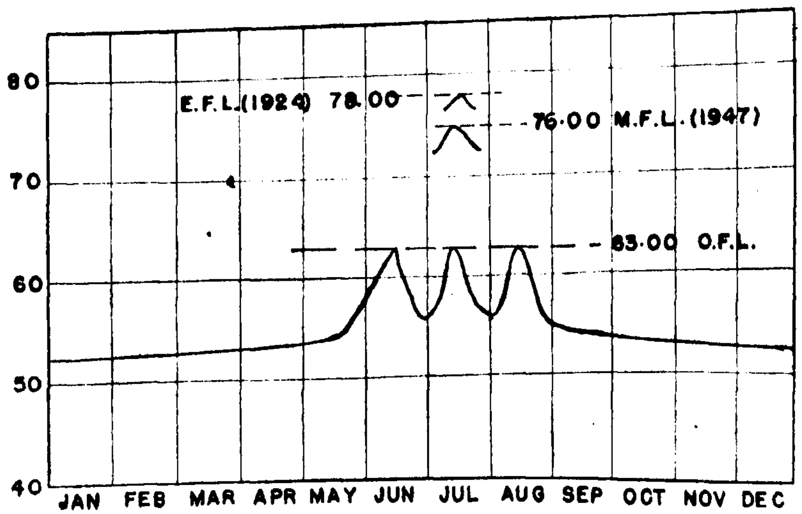


Fig. 2. Flood levels chart at site

4. SELECTION OF SITE

4.1. While selecting a site for a submersible bridge, the following factors should be considered.

The site (i) should have a straight reach on either side well below bends;

(ii) should be situated far away from the confluence of large tributaries;

(iii) should have well defined and fairly high banks;

(iv) should have a small width;

(v) should have good foundation facilities;

(vi) should be such as will afford straight approaches.

4.2. Due to possibilities of outflanking, a straight reach of river and distance from confluence of large tributaries are important. Cutting rather than embankment is to be desired for approaches. So, well-defined high banks are desirable. As scour of bed is particularly more severe in the case of submersible bridges, a site having good foundation facilities is to be chosen. The final selection of site should be such that approaches to the bridge from the existing road can be had economically with straight reaches and curves of large radii.

5. HYDRAULIC PARTICULARS AND DISCHARGE CALCULATIONS

5.1. The need for the correct determination of flood discharge and flood levels is more pronounced in the case of submersible bridges than in the case of high level bridges. The flood discharge of a stream at a particular bridge site is obtained either by the adoption of one or more of empirical formulae, rational formulae or the area-velocity method using the silt factor and slope of the stream. A few of the most popular of these formulae are given below:

5.2. Empirical Formulae

5.2.1. The empirical formulae are based on the catchment area of the stream and are of the form $Q = CM^n$. The most popular is Ryve's formula, originally devised for South India:

$$Q = CM^{2/3}$$

where Q = run-off in cusecs.

M = catchment area in sq. miles.

C = a constant taken as 450 for areas within 15 miles of the coast, 563 for areas between 15 miles and 100 miles of the coast, and 673 for limited areas near the hills.

5.2.2. In this formula much depends on the correct adoption of the value of C.

5.3. Rational Formulae

5.3.1. The rational formulae are relationships recommended by hydrological experts who have tried to establish relationships for flood discharge in terms of intensity, distribution and duration of rainfall and the extent, shape, slope, vegetation, etc., of the catchment. A typical rational formula is as below:

$$Q = A.I_o.\left(\frac{2f\,P.}{t_c+1}\right)$$

where Q = maximum flood discharge in cusecs.

A = catchment area in acres.

I_o = "one hour rainfall" of the area in inches per hour. This can be taken as 4 to 5 inches for areas of intense and prolonged rainfall and 2 to 3 inches for other areas.

t_c = concentration time in hours.

$$= \left(11.9 \times \frac{L^3}{H}\right)^{0.385}$$

where L = the distance from the critical point to the bridge site in miles.

H = the difference in elevation between the critical point and the bridge site in feet.

P = percentage coefficient of run-off for the catchment characteristics, values ranging from 0.9 for bare steep rock to 0.1 for sandy soil with thick vegetation.

and f = a factor to correct the variation of intensity of rainfall I_o over the area of the catchment. Values ranging from 1 for zero area to 0.7 at 30,000 acres and thence to 0.6 at 500,000 acres.

5.3.2. In the above rational method, the values of A, L and H are to be obtained from Govt. of India topographical maps. The value of I_o applicable for the region in question may be obtained from the Meteorological Department of the Govt. of India. Fixation of values of coefficients P and f needs considerable judgment and experience. All the value of laborious calculations will be lost if the above values are not determined correctly, which any engineer who has done investigation for a few bridges would have experienced to be a very tricky job.

5.4. Area-Velocity Method

5.4.1. The area-velocity method is based on the hydraulic characteristics of the stream such as cross-sectional area and slope of the stream, etc. The velocity obtaining in the stream under the flood conditions is calculated and the flood discharge is obtained by multiplying the area of flow by the velocity. The most popular formulae in this category are Kutter's formula and Manning's formula.

5.4.2. **Kutter's formula**:

$$Q = A.\,C\sqrt{RS}$$

where Q = discharge in cusecs.

A = area of flow in sq. ft.

C = Kutter's coefficient

$$= \frac{41.6 + \dfrac{0.0028}{S} + \dfrac{1.811}{n}}{1 + \left(41.6 + \dfrac{0.0028}{S}\right)\dfrac{n}{\sqrt{R}}}$$

S = water slope

n = coefficient of roughness

and R = hydraulic mean radius in feet

$$= \frac{\text{wetted area in sq. feet}}{\text{wetted perimeter in feet}}$$

5.4.3. The area of flow and wetted perimeter are obtained by a scale plotting of the cross section of the stream. Though the water slope is the same as the bed slope in an ideal straight bed, in practice the bed will be found to be undulating and so the value of S is obtained from the difference in the value of M. F. L. at two cross sections, one upstream and one downstream of the section considered. The value of n varies from 0.025 for clean straight stream to 0.050 for sluggish, weedy and stony streams with deep pools. This is fixed with respect to the condition of the stream near the site, again a matter of judgment.

5.4.3. **Manning's formula**:

$$Q = A.\,V.$$

$$= A.\ \frac{1.486}{n}.\ R^{2/3}.\ S^{\frac{1}{2}}$$

with the notations as in 5.4.2.

5.4.4. When the banks are overtopped and in case of wide streams, it would be desirable to split the cross section into smaller areas and calculate discharges for these smaller areas separately and then sum up the individual discharges.

5.5. Flood Levels

Usually considerable difficulty is experienced in the correct determination of the flood levels. Very often this has got to be done only by local observations at the site and local enquiries from people living nearby. This work will be a bit easier if some bridge exists at or near the proposed site, as much valuable information can be had from the performance of that bridge. When making local enquiries, especially in undeveloped areas, an investigator very often comes across conflicting reports, and much would depend on the judgment of the investigator and such a judgment can only be developed by experience. To the experienced investigator, the flood levels are often evident from an inspection of the flood marks and cross section of the stream at site and such marks should be got confirmed by enquiries to the extent possible.

5.6. Discussion of the Formulae for Evaluation of Flood Discharge

The determination of the discharge from a catchment basin at a fixed point is a very complicated matter and there is still no law laid down for its calculation. The empirical and rational formulae have so many constants and variables that various investigators would arrive at different values of discharge for the same site. This is because no two catchments of equal area have the same intensity, duration and frequency of rainfall, shape of the catchment basin, geological formations at and underlying surface and vegetable growth. Further, a lot of personal errors are likely to creep in the matter of fixation of flood levels, classification of catchment and stream for the purpose of choosing values of constants to be used in formulae and correct determination of catchment areas. Catchment areas are generally obtained from Govt. of India topographical maps of scale 1 inch = 1 mile and calculations from these maps cannot be very accurate for small areas. During investigation of bridges near hilly regions in the west coast, the Author has found that the empirical and rational formulae were not very helpful in arriving at the correct flood discharge. For example, in Ryve's formula, the value of C for hilly areas is 673. It has been the Author's experience that the value of C = 673 is a very low figure for hilly areas in South Kanara district in the

West Coast and a value of as much as $C = 1500$ is necessary. The rational formulae, which have been often recommended because they attempt to take into consideration the characteristics of the catchment and rainfall, are too flexible due to possible variations in P and f and possible errors in I_o, L and H. These formulae may, therefore, yield different values of discharges at the hands of different investigators for the same bridge site. This is particularly so in the case of hilly regions where no records of rainfall characteristics are available and sudden and violent floods are common. It would be found that the discharges arrived at by the area-velocity method would be most reliable. For this calculation, we take two cross-sections each at considerable distance upstream and downstream of the site and one cross-section of the stream at the site and compute the average discharge either by Kutter's formula or by Manning's formula. As the process of determination is only approximate, it is felt reasonably accurate to adopt Manning's formula and the use of the graph given by Shri Goverdhan Lal as Plate No.3 in Paper No. 167 of the Indian Roads Congress Journal, Vol. XVIII Part 2 is recommended.

6. AFFLUX

6.1. Afflux is the rise of water level above the normal on the upstream side of a bridge caused by the reduction in waterway due to provision of the bridge. It is measured by the difference in water levels upstream and downstream of the bridge. Afflux will have to be provided for. It can be calculated from the formula :

$$\text{Afflux} = \left[\frac{V^2}{58.6} + 0.05\right]\left[\left(\frac{A}{a}\right)^2 - 1\right]$$

where V = the velocity of approach in feet per second

A = natural area of cross section

a = ventway area.

6.2. Afflux should be as small as possible. The linear waterway is so provided as to include the full bank to bank distance in order to reduce afflux. It may usually be found that for well designed submersible bridges the afflux is generally less than one foot.

7. FIXATION OF VENTWAY

7.1. Discharge through a Submersible Bridge

The discharge in a submersible bridge consists of three parts as shown in Fig. 3.

Total discharge $Q = Q_a + Q_b + Q_c$

$$Q_a = A_a \times \frac{2}{3} \cdot C\sqrt{2g} \cdot \frac{(H + h_a)^{3/2} - h_a^{3/2}}{H} \quad ......(1)$$

$$Q_b = A_b \times C\sqrt{2g} \cdot \sqrt{H + h_a} \quad ......(2)$$

$$Q_c = A_c \times C\sqrt{2g} \cdot \sqrt{H + h_a} \quad ......(3)$$

where h_a = head due to velocity of approach.

$C \doteq 0.625$ for equations (1) and (2)

$C = 0.9$ for equation (3)

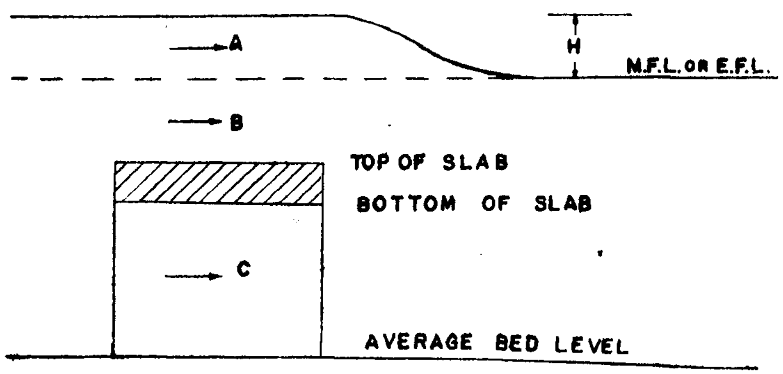


Fig. 3

7.2. For the peculiar conditions obtaining in submersible bridges, slab spans are the most desirable as they offer the least obstruction to the water flow in the submerged condition. Arch spans are not so efficient, though the older submersible bridges were constructed of arches. Pipes are suitable only for small causeways where the ordinary flood is very small.

7.3. To prevent debris blocking the vents, and to reduce the number of piers, spans are to be chosen to be as long as practicable. At the same time, it would be desirable to bear in mind the economical span length, which for slab bridges can be taken as roughly equal to 1.5 H, where H is the total height of

abutment or pier from the bottom of its foundation to its top. Where the bridge is founded on concrete raft resting on sandy bed, the possibility of tension occurring in the concrete raft would further limit the span length. Generally, for submersible bridges founded on rock, slab spans of 20 to 25 feet are adopted.

7.4. In order that the afflux should be small and the velocity in the vents is kept within limits, the discharge per foot width of the bridge should be kept low. The linear waterway is so designed that the O.F.L. discharge could be passed through the vents. It is advisable to allow the full bank to bank waterway without restricting it. Restriction of waterway by guide banks which is common in the case of high level bridges is not suitable for submersible bridges.

7.5. It should also be checked to see that the velocity upstream should not be less than the critical non-silting non-scouring velocity.

7.6. The sides of the stream may be revetted for a short distance on upstream and downstream sides up to 1 foot above E.F.L. where convenient or at least up to 1 foot above O.F.L. A bit of river training done near the site of the submersible bridge will be of advantage to the proper functioning of the bridge.

8. FIXATION OF ROAD LEVEL

8.1. Proper fixation of the road level is very important in the case of submersible bridges and is the prime criterion for the success of the bridge so far as road usage is concerned. Though the I.R.C. Bridge Code permits interruption to traffic for a maximum period of three days at a time for not more than six times in a year, the Author feels it desirable to so arrange the road level that at O.F.L. conditions the water level upstream of the bridge will be just at the road level, if the extra cost due to this arrangement is not very much. It is good practice, therefore, to fix the bottom of slab at O.F.L., so that the thickness of the slab would take care of the afflux that is created. This procedure is eminently suitable for the stream characteristics shown in Figs. 1 and 2. In this case, it would be desirable to have the bottom of slab at R.L. 63.00. The road level would be higher than the bottom of slab level by an amount equal to the thickness of slab and wearing coat.

8.2. However, if the peak of the ordinary flood occurs for a very short while and if that peak is much higher than the succeeding next higher levels, then it may be desirable to fix up the road level lower than the peak level of the ordinary flood.

8.3. In hilly areas where floating trees may be expected during M.F.L. or E.F.L. conditions, it should be checked to see that the road level is sufficiently low to allow the floating trees over the road slab without any danger or damage to the road slab.

9. FORCES ACTING ON A SUBMERSIBLE BRIDGE

9.1. The critical condition for a submersible bridge will be when the water level upstream is just at the road level. Water will be backed up on the upstream side in front of the slab and on the downstream side there will be a trough of standing wave due to the high velocity of the stream.

9.2. The forces acting on the submersible bridge at the critical condition are:

(i) **Pressure due to the static head:** (a) on account of the afflux on the upstream side and the trough of the standing wave on the downstream side.

This is equal to wh

where w = weight of 1 cubic foot of water

and h = afflux + depth of slab and wearing coat + depth of tail water below bottom of slab

This acts to a height from o to h from the surface.

(b) Total pressure due to depth of water $= \dfrac{wH^2}{2}$

where H = height of water on upstream side plus head due to velocity of approach.

This acts on the face of cutwater.

$$\text{Pressure per sq. foot} = \frac{\frac{wH^2}{2}}{\text{area of cut water}}$$

(ii) **Pressure due to the impact of water and debris**

This is given by:

$$\text{Impact pressure} = \frac{wV^2}{2g}\left(\frac{a}{A-a}\right)^2$$

where w = wt. of 1 cubic foot of water

V = velocity of stream

g = acceleration due to gravity

A = unobstructed area of waterway

a = area of obstruction.

(iii) **Pressure due to eddy motion at the back of piers**

$$\text{Pressure due to eddies} = w\,\frac{(V_v - V)^2}{2g}$$

where V_v = velocity of flow through the vents

V = velocity of approach

(iv) **Pressure due to friction of water against piers and bottom of slab**

$$\text{Pressure due to friction} = f.p.\left(\frac{V_v}{10}\right)^2$$

where f = friction coefficient = 1

p = area of surface of submersible bridge in contact with water

V_v = velocity of flow through vents.

(v) **Pressure due to uplift head under the slab**: This acts both horizontally and vertically. It is given by uplift pressure = wh

where h = the uplift head under the decking which is equal to the thickness of the slab and wearing coat and afflux less the head loss due to increase in velocity through the vents.

The head loss due to increase in velocity through the vents

$$= \frac{(V_v^2 - V^2)}{2g}$$

where V_v = velocity in the vents

V = velocity of approach

9.3. The sum total of all the horizontal forces should be taken as acting at one third height of pier to test the stability of the pier against overturning. It would be usually found that this total horizontal force will be very small compared with the net weight of the structure and hence will be safe.

10. SOIL PARTICULARS AND FOUNDATIONS

10.1. Soil Profile

10.1.1. The characteristics of the soil obtaining at the site should be ascertained by trial borings at approximately the abutment and pier locations along the centre line of the bridge. A boring chart showing the soil profile will then be drawn to a suitable scale.

10.1.2. In usual cases, where submersible bridges are to be adopted, it would be found that hard rock or stiff clay is available at shallow depths. In some cases it may happen that a submersible bridge may be necessary at a site where the bed is sandy for a considerable depth. The procedure for design of foundations in the above two cases is slightly different.

10.2. Scour Depth

10.2.1. It is necessary to calculate the deepest scour level. The maximum scour depth may be taken as :

$$D = N.\times 0.473\left(\frac{Q}{f}\right)^{1/3}$$

where D = the depth in feet of the deepest scour level below the flood level considered.

Q = discharge in cusecs for the scour level considered

f = silt factor which is equal to 1.25 for medium sand, 1.5 for coarse sand, 2.0 for bajri and about 5 for gravel

N = a constant equal to 2 in the case of scour at noses of piers and 1.5 in the case of scour at abutments.

10.2.2. It should be noted that though the vent area is designed for O.F.L. conditions, the scour depth should be calculated taking into account the worst effect of the floods that may occur at the site.

10.3. Foundations in Hard Rock

When hard rock is available at shallow depths above the calculated scour level, the substructure should be secured to the rock by means of two or more rows of dowel bars of, say, 1½ in. dia. at about 4 feet centres. These dowel bars may extend to about 3 feet into the rock and the space between the rod and the sides of the hole may be filled with molten lead. The length of the rod in the substructure may be at least 3 feet including a hook of 1 foot 1½ inches.

10.4. Foundations in Hard Soils other than Rock

In case a hard soil other than rock, which is not erodible at the velocity obtaining at the site, is met with above the calculated scour level, the foundations can be rested in such hard material with a grip length of about three to four feet.

10.5. Foundations in Sand

10.5 1. Sandy strata is generally not a favourable condition for submersible bridges. If the sand strata extends for considerable depth, resort is usually made to a monolithic base construction or a cement concrete raft which runs continuously over the entire length. Sloped aprons and cut-off walls on both upstream and downstream sides are provided to guard against scour and undermining, which are severe in the case of submersible bridges. Boulder pitching with large diameter boulders is provided in the bed on either side beyond the cut-off walls.

10.5.2. Cement concrete raft

The concrete raft serves to distribute pressure evenly to the sand surface and also the arch action helps in taking up the soil pressure. As this raft is usually unreinforced, there should be no tension induced in it. This condition restricts the span length of the vents and large rectangular openings are not possible. One precaution to be adopted is that the floor should be kept at some depth below the low bed level to prevent the floor acting as a weir when retrogression of levels takes place.

10.5.3. Cut-off walls and impervious aprons

The lengths and thicknesses of cut off walls and impervious aprons are calculated just as would be done in the case of weirs having the height of top of shutter as the height of water on upstream side at the critical condition.

There are three methods of determining the lengths of cut-off walls and impervious aprons, viz:

(a) Bligh's creep theory

(b) Lane's weighted creep theory

and (c) Khosla's method of independent variables applied to potential theory.

While it will be beyond the scope of this Paper to explain all the above three theories, it would be worth while to mention the salient points useful for the design of cut-off walls and aprons.

By Bligh's creep theory, the creep length

$$CH = L + 2\,(d_1 + d_2)$$

where H = pressure head, taken as equal to depth of water on upstream side including afflux

L = Length of impervious apron

d_1 = depth of upstream cut-off wall

d_2 = depth of downstream cut-off wall

C = a constant called creep coefficient, values of which for different soils are as given in Table 1.

According to Bligh's theory, vertical cut-offs are twice as effective as horizontal aprons in contributing to creep length and the rate of head loss along the creep length is assumed to be linear. A number of structures throughout the world were built as per Bligh's theory and while some stood well, others have failed, making the theory inapplicable now.

Lane conducted a statistical study of 270 actual structures on permeable foundations, both sound ones and failures, and arrived at the conclusion that greater weightage should be given to the vertical cut-offs than horizontal aprons and suggested a ratio of 3 to 1. As per his weighted creep theory,

$$C_1\,H = 1/3\,L + (d_1 + d_2).$$

where C_1 = weighted creep coefficient as given in Table 1.

TABLE 1

Creep coefficients

Material	C_1 Lane's weighted creep coefficient	C Bligh's creep coefficient
Very fine sand or silt	8.5	18
Fine sand	7.0	15
Coarse sand	5.0	12
Gravel and sand	3.0 to 3.5	9
Boulders, gravel and sand	2.5 to 3.0	9
Clayey soils	1.6 to 3.0	4 to 6

Lane's approach, though an improvement over Bligh's theory, has still the limitations of an empirical method. However, it can be used as an alternative in testing designs.

When the subsoil is homogeneous, Khosla's method of independent variables applied to the potential theory is the most reliable. Potential theory is applicable to systems of flow which satisfy Laplace's Potential Equation and subsoil flow satisfies this equation.

$$\text{Let } \frac{L}{d_2} = \alpha$$

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2}$$

$$\text{Exit gradient} = \frac{H}{\pi\, d_2 \sqrt{\lambda}}$$

To safeguard against undermining, the exit gradient should not exceed the numerical value of unity. Providing a factor of safety of 5 to 7, the dimensions of the cut-offs and apron should be so chosen that the exit gradient is limited to 1/5 to 1/7.

To find the necessary thickness of apron, the uplift pressures at salient points are calculated and the thickness necessary to counter this uplift pressure is determined. As it is beyond the scope of this Paper to explain the subsoil flow theories fully for a complete treatment of this aspect of design, attention is drawn to Khosla's work on "Design of Weirs on Permeable Foundations", Central Board of Irrigation Publication No. 12.

10.5.4. Beyond the impervious apron, talus aprons are provided for short lengths on either side to keep the scour holes at safe distance. The ends of the talus aprons may be protected with retaining walls. The diameters of boulders required for pitching can be obtained by Chailly's formula, which gives the diameter of stones that can be moved by velocity as:

$$V = 5.67\ \text{G.d.}$$

where V = velocity of stream in feet per second

G = specific gravity of stone

d = diameter of boulder in feet

This can also be written as

$$d = \frac{V^2}{85}.$$

Allowing a factor of safety of 2, the minimum diameter of boulders required for pitching can be taken as $d = \frac{V^2}{42.5}$. The values of boulder sizes necessary for different velocities are listed in Table2.

TABLE 2

Minimum diameter of boulders required for pitching

Velocity in feet per second	Diameter of boulders required for pitching in feet *
0 to 6.0	1.0 (nominal)
7.0	1.5
10.0	2.75
12.0	4.0
15.0	6.0

10.6. In all the above cases, the pressure at the base of the foundation should be well within the safe bearing capacity of the foundation material.

10.7. From the foregoing it will be evident that a submersible bridge on a sand bed will be very costly due to the special nature of foundations required. So the comparative cost for a high level bridge with individual footing foundations and for a submersible bridge should be carefully studied. It will be often found that, where sandy beds are encountered, the difference between O. F. L. and E. F. L. may not be very much, thus tipping the scale in favour of a high level bridge.

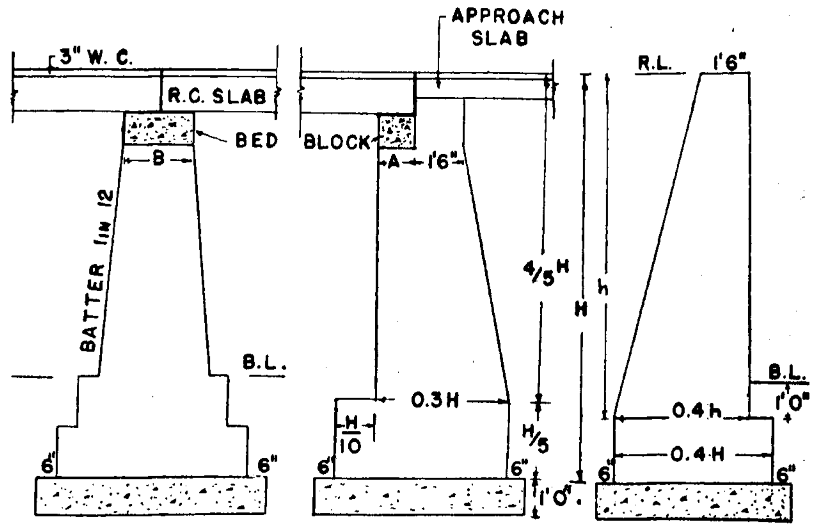
11. ABUTMENTS AND PIERS

11.1. Abutments and piers may be constructed either of cement concrete or masonry. In most places concrete construction may be economical. This fact is attributable to the large labour cost involved in construction of masonry, the present scarcity of skilled masons and the non-availability at many localities of good quality stone except at a prohibitive cost where transportation is an adverse factor. With mass concrete construction, less skilled labour can be used and handling and

* For values larger than 2.0 feet, boulder pitching encased in wire netting is provided.

transportation of all material will be easier. In some localities it may be found advantageous to use a masonry facing and concrete backing, the advantages being saving in the expense of forms, quicker construction, attractive appearance and elimination of surface cracks. In remote localities and in community project areas, where submersible bridges will be used most, mass concrete construction may be found favourable. If the substructure is of masonry, coursed rubble masonry in cement mortar 1 : 5 may be adopted. With mass concrete, cement concrete 1 : 3 : 6 mix using 1½ in. size granite metal may be used. If desired, further economy may be achieved by using the core plums of large clean and sound boulders placed by hand not closer than 1 foot in all directions, to an extent not exceeding 20 per cent of the total volume of coarse aggregates.

11.2.1. The sections of abutments and piers for submersible bridges may be same as those for high level bridges of equal height. As the spans are usually slab spans of 25 feet and less and the heights are also less, they do not merit laborious calculations. The sections given in Fig. 4 are quite suitable.



(a) PIER (b) ABUTMENT (C) WING

Fig. 4. Substructure details

11.2.2. The top width of piers may be taken as twice the bearing width for each slab; for abutments, it equals the bearing width of one slab plus 1 foot 6 inches for the back wall. In the case of wing walls, the top width is taken as 1 ft 6 in. The final width of base concrete should be such that the pressure on soil is within permissible limits. For wings, rectangular returns to a length from face of abutments of 1.5 times the height from road level to bed level may be preferred to splayed wings, as the former adds to the stability of the structure during submersion.

11.3. Semi-circular cutwaters may be provided for piers and the corners of the face of the abutments may be rounded to increase efficiency of discharge.

11.4. Bed block of 1 ft 0 in. thickness with cement concrete 1 : 3 : 6 mix using $\frac{3}{4}$ in. granite metal may be provided over the piers and abutments. This, besides providing an even surface support, would also aid in providing camber and in housing the pipes for deck slab anchorage.

11.5. R.C. coping on the cutwaters of piers would add to the appearance and also aid in the streamlined flow of water.

12. DECK SLAB

12.1. The deck slab for submersible bridges may be of the same thickness as in the case of high level bridges. The slabs will be designed for I.R.C. Class 'A' two lane loading. The clear roadway between kerbs or guard stones should be kept as 24 ft 0 in.

12.2. The upstream and downstream edges of the deck slab are provided with semi-circular horizontal cutwaters to enhance the discharging efficiency of the slab during overtopping. The distributor rods are bent suitably to conform to the cut water shape.

12.3. The thickness and reinforcements for deck slabs should conform to the Tables given in Plate I. The concrete used will be cement concrete 1 : 2 : 4 mix ordinary grade, using $\frac{3}{4}$ in. size granite metal.

12.4. In submersible bridges, simple spans are to be preferred. Though adoption of slabs continuous over three spans would give some saving in cost due to smaller thickness of slab, it has some disadvantages. Firstly, on submergence, the weight of slab will be less, an unfavourable condition for stability. Secondly, due to non uniform reaction

at the supports, there is a grave danger to the structure in the event of any unequal settlement. So, especially for submersible bridges founded on soils other than rock, continuous slab construction is not advisable.

12.5. **Wearing Coat**: A wearing coat of 3 in. uniform thickness will be provided over the deck slab, with cement concrete 1 : $1\frac{1}{2}$: 3 mix using 65 per cent of 1 in. size, 25 per cent of $\frac{1}{2}$ in. size and 10 per cent of $\frac{1}{4}$ in. size hard granite metal.

12.6. **Bearings**: As the spans are usually slab spans of less than 25 feet, and as submergence in water will be frequent, tar paper will be provided between slab and bed block.

12.7. **One-way Camber**:

12.7.1. Generally camber is provided on both sides of the centre line of the road slab. Experiments [3] have indicated that one-way camber towards downstream side is advantageous in that this arrangement reduces afflux and raises upstream edge, keeping the roadway dry up to a higher depth of streams on the upstream side.

12.7.2. The one-way camber is conveniently formed by adjusting the levels of the bed block suitably. A camber of 1 in 72 will be adequate.

13. ANCHORAGE OF DECK SLAB TO SUBSTRUCTURE

13.1. In the case of a submersible bridge, the deck slab is near the plane of maximum velocity. To counteract the sliding action due to velocity of flow, loss of weight of slab due to buoyancy, the tilting forces due to eddies and currents and the disturbing forces due to debris or trees floating down the stream, it is necessary to anchor the deck slab to the substructure.

13.2. One possible solution to this anchorage is as shown in Fig. 5. The aim in this anchorage is to secure the deck slab to the piers or abutments against uplift or lateral thrust and at the same time allow lateral movement due to expansion and contraction due to temperature effects. The arrangement will be evident from the figure.

14. KERBS AND GUIDE POSTS

14.1. The general tendency is to discourage the use of kerbs and guide posts on submersible bridges, but only to allow guard

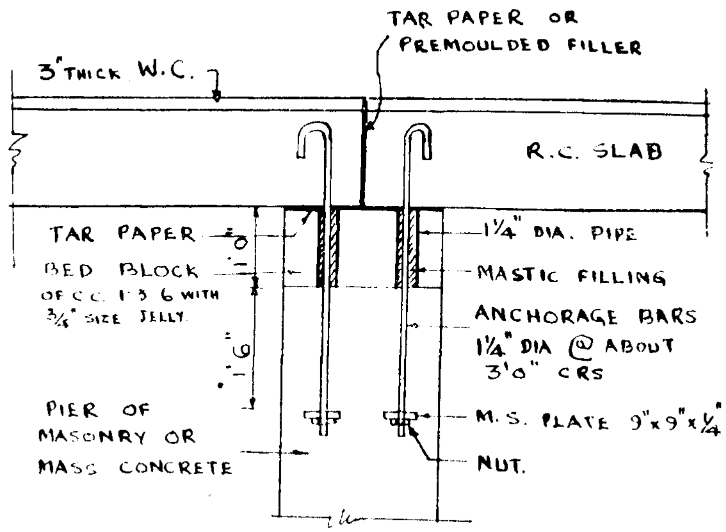


Fig. 5. **Deck slab anchorage**

stones of various shapes including streamlined guard stones. Still, provision of kerbs will be desirable in view of the additional safety afforded to vehicles, specially bullock carts using the bridge, particularly during nights. If as suggested in para 8.1, the bottom of slab is kept at O.F.L., the small obstruction to flow that may be caused by the kerbs, which may be perforated as explained below, will be infrequent and negligible.

14.2.1. Two types of R.C. kerbs may be adopted as shown in Plate II, where typical details are given for a slab for clear span of 25 feet.

14.2.2. The first type will be of perforated type, consisting of rails and posts of small height. The number and size of gaps may be varied to suit the lengths of slabs. Typical details of reinforcement of perforated kerbs for a clear span of 25 ft are given in the plate. The appearance of this type of kerbs will be pleasing. It strikes the golden mean of providing a pleasing wheel-guard at the same time satisfying the functional requirement of allowing discharge of water.

14.2.3. The second type consists of R.C. blocks of 9 in. height as indicated in the plate. The outer side is made triangular to get streamlined effect. The size and spacings of blocks may be varied to suit individual cases. This type is more sturdy and can be adopted where a certain amount of floating debris at O.F.L. is expected.

14.3. Streamlined guard stones and nearly conical R.C. wheel-guards have been recommended for submersible bridges. These may be used under adverse conditions when the difference between O.F.L. and E.F.L. is small and when floating trees are expected at E.F.L. under such conditions or lower levels.

14.4. Where conditions are favourable, such as considerable difference between O.F.L. and E.F.L., infrequent occurrence of floating trees and lower velocity of flow, diamond shaped R.C. guide posts would be desirable. This type of guide posts would be particularly suited with R.C. perforated kerbs. Typical details of reinforcements of such guide posts are shown in Plate II. End posts will be omitted in alternate slabs to avoid double posts.

14.5. Collapsible railings, though ideal if proper maintenance and timely action in releasing pins could be guaranteed, may not be fool-proof in actual practice. If they are not properly maintained, they may give more trouble during floods.

15. APPROACHES

15.1. It is desirable to have the approaches in cutting as embankments are liable to be washed away in each flood. In cutting there may be the problem of silting. Silting can be appreciably reduced if the approaches are aligned at a slight angle to the centre line of the bridge so that the gradient falls in the direction of the river. In actual practice, the amount of silting in a well designed submersible bridge with straight approaches may not be very much. On the other hand, straight approach is always preferable from the point of view of traffic and also ease in construction of wing walls.

15.2. The sides of cutting should be protected by rough stone revetment up to about 1 foot above E.F.L. This would avoid scouring of the sides of cutting and consequent silting of the approaches.

15.3. It is desirable to give a flat gradient not steeper than 1 in 30 or at the most 1 in 20 for at least 50 feet on either side of the bridge where the site conditions permit. Beyond this, steeper gradients up to 1 in 15 may be used for short lengths.

15.4. An approach slab 6 in. thick with R.C. 1:2:4 mix using $\frac{3}{4}$ in. metal and having reinforcements of $\frac{1}{2}$ in. diameter rods at 6 in. centres bothways at the top and bottom may be provided for about 20 feet length on either side of the bridge. A 6 in. thick soling may be provided below the approach slab. 3 in. thick wearing coat may be provided on the approach slabs as well.

16. NEED FOR FURTHER RESEARCH

16.1. Though quite a number of submersible bridges have been built in India, no enthusiastic attempt has been made to record scientific details about their performance and the effect of various components of the bridge on the overall efficiency of the structure. Some research[3] has been made (i) on the effect of the design of causeways on the river course and on the flow of water inside the vents and over the roadway and (ii) on the effects of the conditions of flow. The research on these points may be continued.

16.2. Some of the other points which need further research are suggested below:

(i) Comparative economy of submersible bridges over high level bridges and effects on short term and long term economy.

(ii) Types of anchorages to be adopted between slab and substructure.

(iii) Types of kerbs and railings suitable for various conditions obtaining at the site, both from the point of view of traffic safety and from the point of view of overtopping during floods.

(iv) Types of abutments and necessity or otherwise of wings and their shapes.

(v) Desirability and extent of inclined alignment of approaches with respect to centre line of the bridge, both from the point of view of avoiding silting and from the point of view of traffic safety and ease of construction of wings. The nature of transition from the straight centre line of the bridge to the inclined centre line of the approaches is also to be studied.

(vi) Effects of decreasing the linear waterway by provision of guide banks.

(vii) Design of foundations in sandy beds, with particular reference to:

(a) design of cut-off walls and impervious apron

(b) provision of talus aprons

(c) provision of raft versus pile or well foundations.

In this connection it may be noted that foundations claim a major share of the total cost in the case of submersible bridges rested on sandy soils. It is seen that the design of impervious apron as recommended at present is based on Khosla's theory, which is primarily meant for weirs. While in the case of weirs the difference in head is considerably great, there being water only on the upstream side and no tail water on downstream side in the critical case, the difference in head in the case of a submersible bridge is always comparatively less as the tail water level is close to upstream water level. If this point is taken into consideration, it is felt that the length of impervious apron required should work out to be less than that for a weir. Further experimental research is necessary to effect economy in this regard.

17. CONCLUSION

From the above, it will be seen, that the subject of submersible bridges deserves greater attention from highway engineers than has been given in the past. Within its limitations, the submersible bridge offers a cheap solution for a crossing on roads of secondary importance. The investigation of a submersible bridge calls for a greater detailed attention in the field and a statement of the particulars to be gathered during investigation is enclosed as Appendix. The peculiar conditions obtaining in the case of submersible bridges need further research.

ACKNOWLEDGMENT

The references cited below have been consulted freely and are gratefully acknowledged. The Author records his gratitude to Mrs. L. Johnson and Dr. A. Nallathambi of Ambur for their help and encouragement.

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Appendix

Statement of particulars to be gathered during investigation for submersible bridges.

1. Location:
   - (i) Name of the bridge and/or mileage and name of the road
   - (ii) Name of the stream
2. Details of the road:
   - (i) Classification and ownership
   - (ii) Type of the road and its present condition
   - (iii) Traffic intensity on the road (Tons—daily)
     - (a) Average— (i) Iron tyred
       (ii) Pneumatic tyred
     - (b) Peak— (Season to be specified)
       (i) Iron tyred
       (ii) Pneumatic tyred
   - (iv) Importance of the road
   - (v) Urgency of the scheme and benefits available by constructing the bridge
3. Details of the stream:
   - (i) Whether the stream is perennial or seasonal
   - (ii) Origin of the stream and approximately the distance from the origin and approximately the difference in elevation between the origin and the bridge site
   - (iii) Nature of the stream near bridge site—meandering, straight, bends, presence of weeds, pools, etc.
   - (iv) Whether the stream serves at present inland water transport
4. Catchment:
   - (i) Area. (Source of the information to be specified and a map to be enclosed)
   - (ii) Permeability of the soil and vegetable cover
   - (iii) Shape
   - (iv) Whether sudden floods are expected
   - (v) Value of C in Ryve's formula appropriate for the catchment with reasons

(iv) Brick

(v) Sand

(vi) Water

(vii) Timber and other materials

20. Labour:

Whether skilled, semi-skilled and unskilled labour required for the work is available locally and, if not, wherefrom they be imported.

21. Machinery and Plant:

List of machinery and plant required

22. Rough cost of the scheme:

(i) Rough cost for a submersible bridge

(ii) Rough cost for a high level bridge

23. Drawings to be enclosed with this report:

(i) Key map, scale 1 inch = 4 miles

(ii) Index map, scale 1 inch = 1 mile

(iii) Site plan, scale 1 inch = 100 feet

(iv) Catchment map, scale 1 inch = 1 mile

(v) Cross sections of the stream, scale 1 inch = 20 feet (natural)

(vi) Longitudinal section of approaches to extend beyond the limits where the new approaches would join the existing road

Scale: Horizontal 1 inch = 100 feet

Vertical 1 inch = 20 feet

(vii) Cross sections of the approaches at every 200 feet scale 1 inch = 20 feet (natural)

(viii) Boring chart showing subsoil profile

Scale: Horizontal 1 inch = 20 feet

Vertical : 1 inch = 10 feet or 20 feet

5. Rainfall:

(i) Intensity

(ii) Distribution in time and space

(iii) Duration

6. Cross sections of the stream:

(i) Cross sections should be taken at—

(a) the proposed site
(b) 330 feet upstream of the proposed site
(c) 660 feet upstream of the proposed site
(d) 330 feet downstream of the proposed site
(e) 660 feet downstream of the proposed site

For major bridges cross sections should be taken for longer distances.

(ii) Cross sections should include the following markings:

(a) Low water level
(b) Ordinary flood level
(c) Maximum flood level with years of occurences
(d) Extraordinary flood level with the year of occurrence

(iii) A short note as to how the levels in (ii) have been obtained and their reliability

(iv) Whether floating trees are to be expected and at what levels of flow

7. Calculated discharge:

(i) By Ryve's formula

(ii) By Manning's formula

(iii) Design discharge

8. Flood levels chart

9. Nature of subsoil

Boring chart drawn to scale is to be enclosed

10. Maximum scour level

11. Kind of bridge necessary and reasons for its choice

12. Bottom of slab level:

(i) Bottom of slab level proposed

(ii) Duration of interruption to traffic

(iii) Reasons for fixation of the level in (i)

13. Lineal waterway:

(i) Lineal waterway proposed with reasons for adoption of the span length proposed

(ii) Bank to bank distance at O.F.L.

14. Foundations:

(i) Type of foundation proposed

(ii) If founded on rock, type of anchorage of substructure to foundation proposed, if any

(iii) If founded on sandy soils, details of foundations proposed, with particular reference to aprons, cut-off walls, etc.

15. Superstructure:

(i) Deck slab

(a) thickness and reinforcements
(b) clear roadway
(c) loading adopted

(ii) Kerbs and guide posts or guard stones, if any.

(iii) Camber

(iv) Wearing coat

(v) Expansion joints

16. Substructure:

Pier and abutment sections and material proposed.

17. Approaches:

(i) Approach slab, length

(ii) Nature of approaches, cutting or embankments, gradients, lengths, etc.

(iii) Extent of revetment necessary for approaches

18. Miscellaneous:

(i) River training, extent necessary

(ii) Working seasons during the year

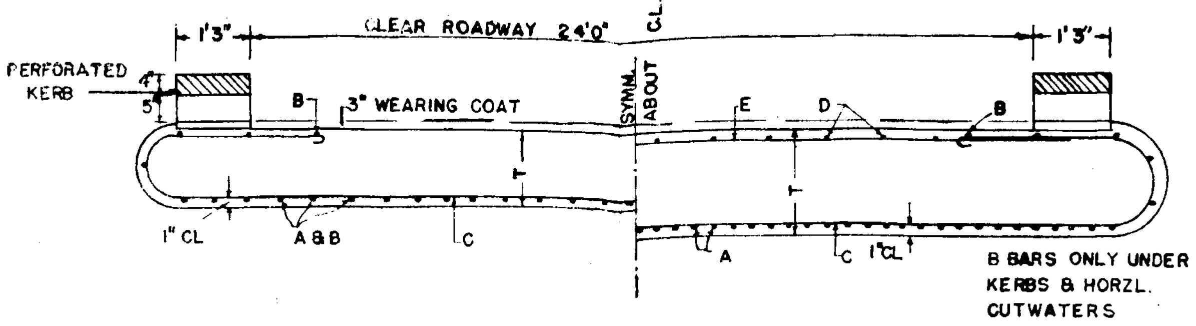
19. Materials:

Quality and "Lead" for materials, i.e., the distance and mode of conveyance to the site from the nearest place of availabilily of

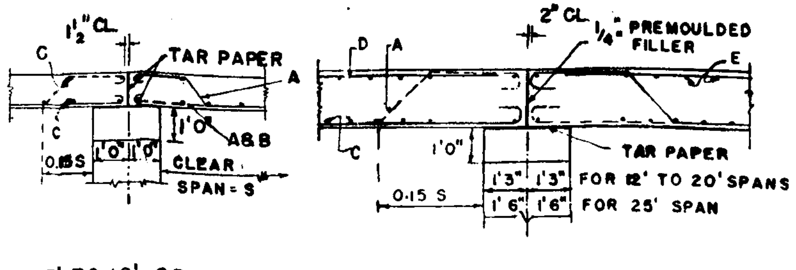
(i) Cement

(ii) Steel

(iii) Stone (for masonry and concrete)



TYPICAL CROSS SECTION



PIER DETAILS

REINFORCEMENT ⊕

	A									B					
SPAN	NO.	SIZE	SPACING	F	G	H	J	LENGTH	NO.	SIZE	SPACING	K	L	LENGTH
5'	22	⅝"	13"	5'1½"	6¼"	1'3"	6"	7'10¾"	29	⅝"	13"	6'9"	6"	7'9"
8'	24	¾"	12"	7'7½"	11"	1'5¾"	7"	11'2¼"	31	¾"	12"	9'9"	7"	10'11"
10'	29	¾"	10"	9'4½"	1'1"	1'7¼"	7"	13'2¾"	36	¾"	10"	11'9"	7"	12'11"
12'	65	¾"	4½"	11'4"	1'4"	1'10¾"	7"	15'8¾"	8	¾"		14'2"	7"	15'4"
15'	45	1"	6½"	13'10"	1'6¼"	2'3"	9"	19'1¼"	8	1"		17'2"	9"	18'8"
18'	53	1"	5½"	16'4"	1'9¾"	2'7"	9"	22'2¼"	8	1"		20'2"	9"	21'8"
20'	58	1"	5"	18'1"	1'11¼"	2'9½"	9"	24'2¼"	8	1"		22'2"	9"	23'8"
25'	72	1"	4"	22'7"	2'8½"	5'2"	9"	29'11½"	8	1"		27'8"	9"	29'2"

	C								D				E				SLAB THICKNESS 'T'	ESTIMATED QUANTITIES	
SPAN	NO.	SIZE	SPACING	M	N	O	P	LENGTH	NO.	SIZE	SPACING	LENGTH	NO.	SIZE	SPACING	LENGTH		CONCRETE CFT.	REINFT. LBS.
5'	13	½"	10"	26'6"	8"	1'10½"	4½"	32'4"									7"	112	710
8'	14	⅝"	12"	26'6"	1'1½"	2'4½"	6"	34'6"									10½"	242	1420
10'	16	⅝"	11"	26'6"	1'3¾"	2'4½"	6"	34'10½"									12"	333	1860
12'	18	⅝"	10½"	26'6"	1'7"	2'4½"	6"	35'5"	18	½"	15"	14'0"	13	½"	15"	25'6"	14"	472	2790
15'	24	⅝"	8½"	26'6"	1'10"	2'4¼"	6"	35'11"	23	½"	12"	16'3"	19	½"	12"	25'6"	16"	654	4150
18'	31	⅝"	7½"	26'6"	2'1"	2'4½"	6"	36'5"	26	½"	10½"	18'9"	25	½"	10½"	25'6"	18"	864	5550
20'	35	⅝"	7"	26'6"	2'3½"	2'4½"	6"	36'10"	26	½"	10½"	20'6"	27	½"	10½"	25'6"	19½"	1 030	6450
25'	31	¾"	12"	26'6"	3'0"	2'10"	7"	59'4"	30	½"	9"	25'3"	31	½"	9"	25'7"	25"	1 666	7770

⊕ BAR SHAPES AS BELOW

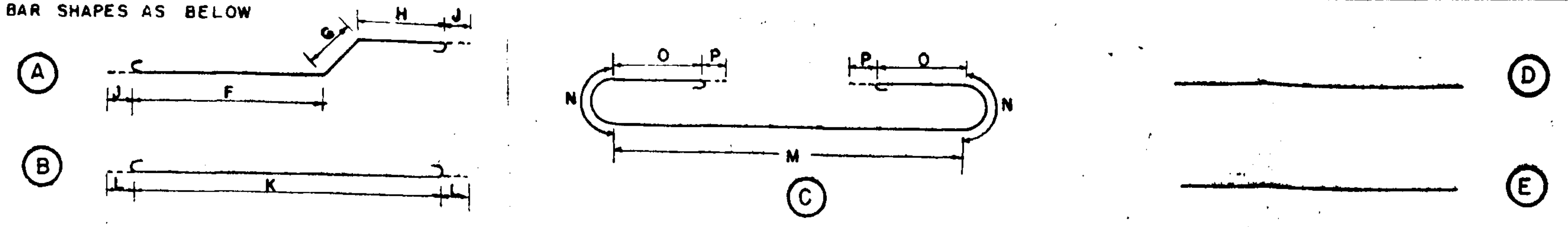
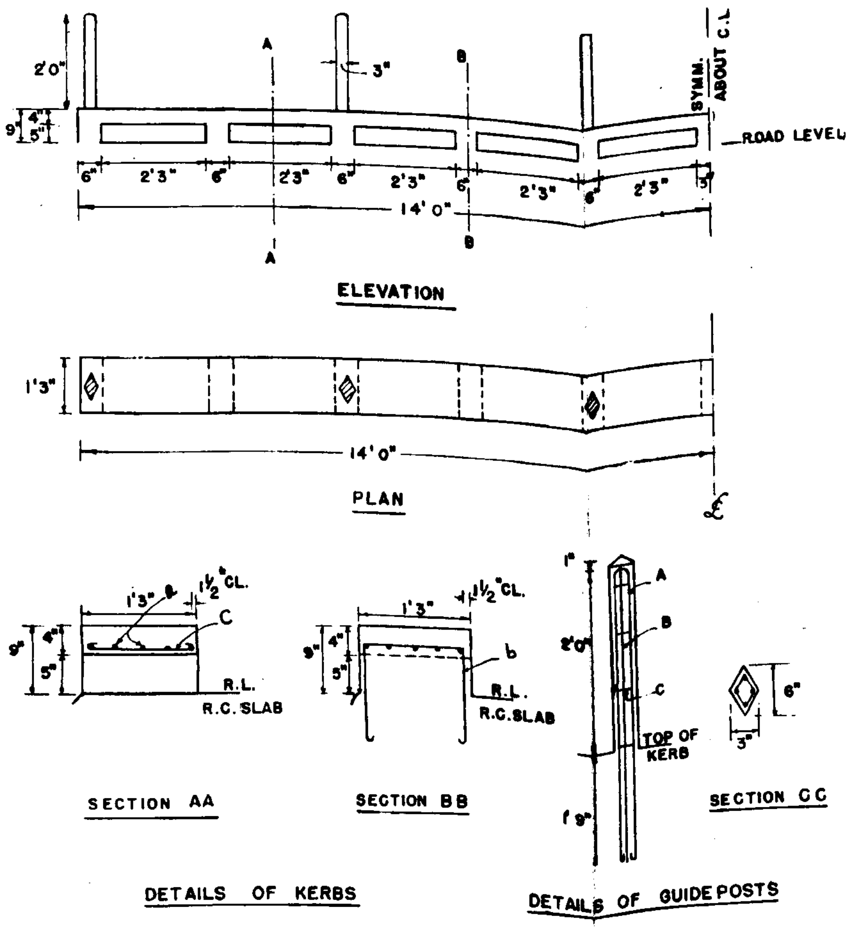


PLATE I

NOTES

1. DESIGN LOADING: I. R. C. Standard Class 'A' (1948) Wheel Load Train. Double Lane.
2. STRESSES: Concrete—Ordinary grade 1: 2: 4 mix
   f_c = 750 lb per sq. in.
   STEEL—B. S. S. No. 785 (1938) or equivalen
   I. S. S.
   f_s = 18,000 lb per sq. in.
3. MODULAR RATIO: m = 15, assumed as fixed.
4. Spacing of distributors given in the Table applies to middl portion of the slab.
5. Distribution rods should be securely welded or wired t the main rods at every alternate joint.
6. For slabs of spans 12 feet and above, nominal surfac reinforcements are provided at top to countera temperature effects.
7. When full length rods are not available and joints are t be provided, lap joints of 45 times diameter are permi sible, provided no joint comes at midspan and all joint are well staggered.
8. Quantity of concrete given is exclusive of wearing co and kerbs and guide posts.
9. Quantity of reinforcement given is exclusive of allowan for splicing and wastage.
10. One way camber will be provided by making adjustmen in the bed block.
11. Sketches shown are not to scale.

REINFORCEMENT DETAILS FOR R.C. SLABS FOR SUBMERSIBLE BRIDGES



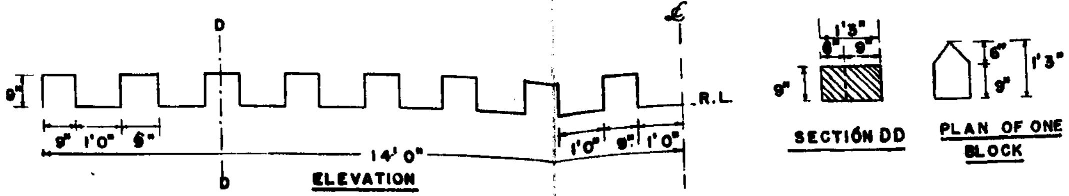
R.C. PERFORATED KERBS & GUIDEPOSTS

STEEL REQUIREMENTS PER SPAN FOR PERFORATED KERBS

NAME	NO.	SIZE	LENGTH	WEIGHT LBS.	SHAPE
a	10	½"	28'6"	196	4½" 27'3" 4½"
b	22	½"	3'5½"	53	12½" 10" 4½" 4½"
c	40	¼"	1'9½"	12	4½" 12½" 4½"
				261	
ADD 5% FOR SPLICINGS WASTAGE				13	
ADD FOR BINDING WIRE				3	
			TOTAL	277	

STEEL REQUIREMENTS PER POST FOR GUIDE POSTS

NAME	NO.	SIZE	LENGTH	WEIGHT LBS.	SHAPE
A	1	½"	8'5"	6	6" 3½" 4½" 4½"
B	1	½"	8'2"	6	3" 3½" 4½" 4½"
C	4	¼"	1'6"	1	3½"
				13	
ADD FOR SPLICING, WASTAGE & BINDING WIRE				1	
			TOTAL	14	



KERB WITH R. C. BLOCKS

KERBS & GUIDE POSTS FOR SUBMERSIBLE BRIDGES

"RECOMMENDED PRACTICE FOR NUMBERING BRIDGES AND CULVERTS"

1. INTRODUCTION

1.1. Considering that a uniform system of numbering culverts and bridges on roads is desirable, the Specifications and Standards Committee of the Indian Roads Congress has recommended a standard practice for numbering such structures.

2. SCOPE

2.1. The standard practice should be adopted on all roads in the country.

3. METHOD

3.1. Cross-drainage works shall be numbered separately in each mile of a road.

3.2. A cross-drainage work shall be designated by a fraction in which the number of the mile, in which the structure is situated shall be the numerator and the mile-wise serial number of the structure the denominator. For instance, the 5th culvert in the fourth mile (between milestones 3 and 4) shall be designated as $\frac{4}{5}$.

3.3. If any new culverts are built, say, between the 3rd and 4th culverts in mile 375, the new culverts shall be designated as $\frac{375}{3-1}$, $\frac{375}{3-2}$, etc.

3.4. The number of a culvert shall be inscribed near the top of the left-hand side parapet wall as viewed in end elevation so as to face the traffic from each direction (see Plate I).

If the structure has railings without end pillars, separate pillars shall be put up for inscribing the structure number.

3.5. The numerals shall be in the international form of Indian numerals and of the size and shape as shown in Plate II. Numerals shall be painted in black on a smooth panel 30 cm × 30 cm in size. The colour of the background shall be canary yellow, I.S.I. shade No. 309.

4. CHANGE TO THE METRIC SYSTEM

4.1. When kilometer stones are fixed on any road the kilometer number shall replace the mile number in the numerator, and the serial number of the structure in that kilometer shall be the denominator.

PLATE I

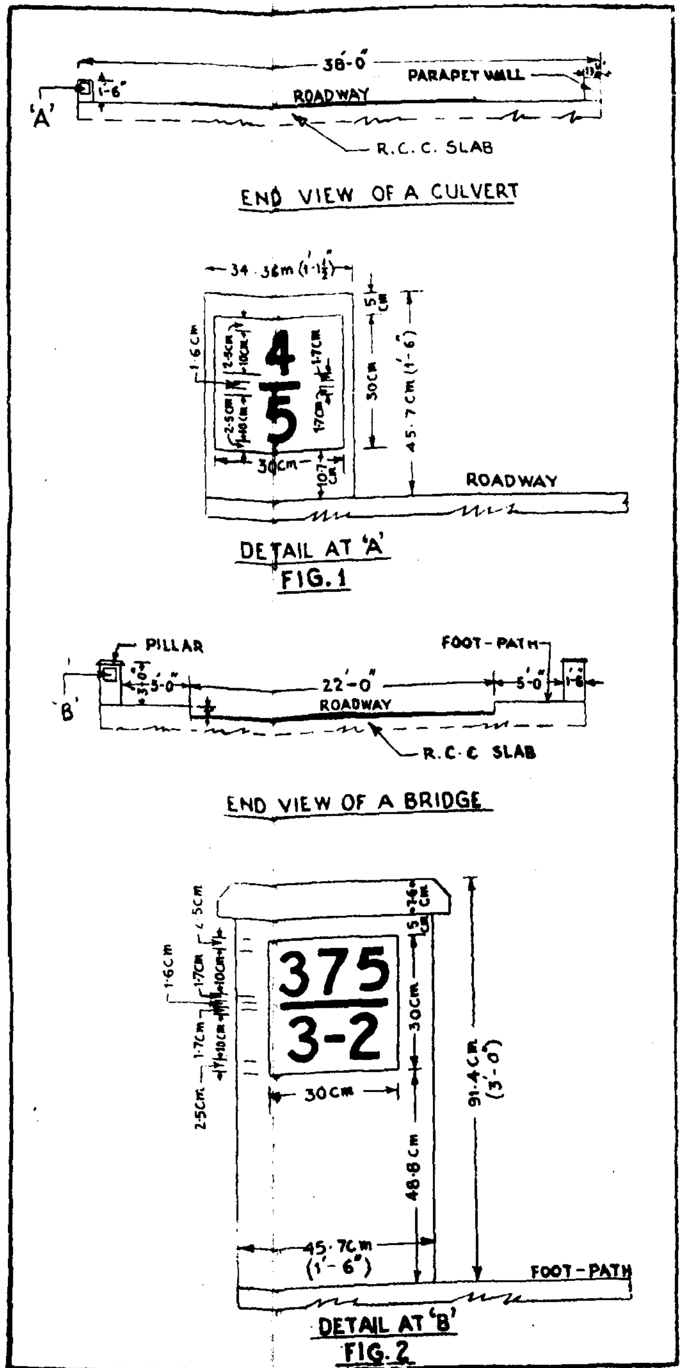
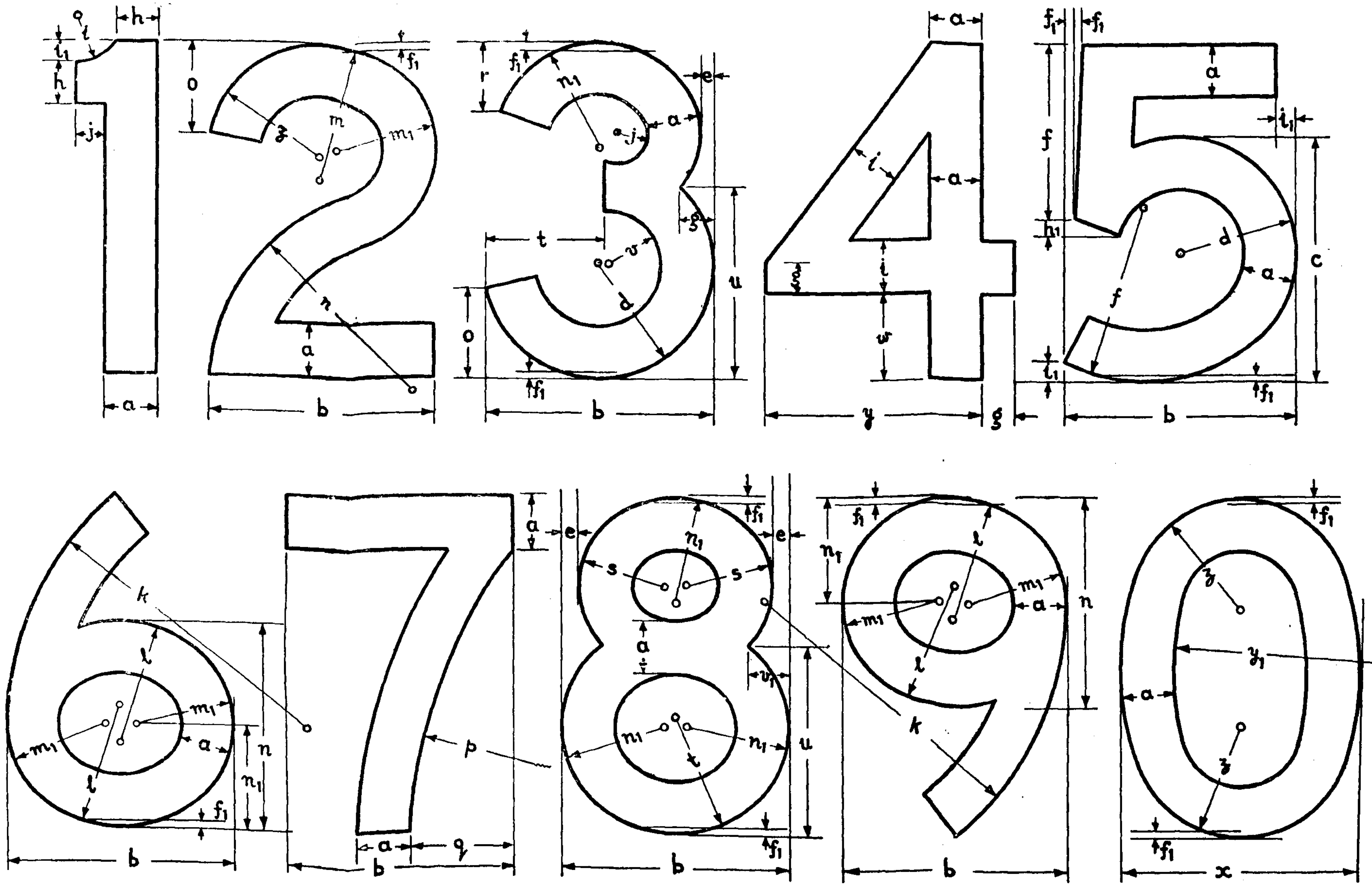

38'-0"
PARAPET WALL
ROADWAY
'A'
1'-6"
R.C.C. SLAB
END VIEW OF A CULVERT
34.38cm (1'-1½")
4
5
30cm
ROADWAY
45.7cm (1'-6")
DETAIL AT 'A'
FIG. 1
PILLAR
FOOT-PATH
'B'
5'-0"
22'-0"
ROADWAY
R.C.C. SLAB
END VIEW OF A BRIDGE
375
3-2
30cm
48.8cm
91.4cm (3'-0")
45.7cm (1'-6")
FOOT-PATH
DETAIL AT 'B'
FIG. 2

PLATE II



DIMENSIONS (MILLIMETERS) FOR 10 CENTIMETER HIGH NUMERALS

a	b	c	d	e	f	f_1	g	h	h_1	i
16	68	73	34	4	52	2	10	13	5	14
i_1	j	k	l	m	m_1	n	n_1	o	p	q
6	9	90	37	40	29	62	31	27	152	30
r	s	t	u	v	v_1	w	x	y	y_1	z
21	26	35	57	15	12	25	71	65	102	33

KEY

SPACING FOR NUMERALS

NUMERALS	LEFT	RIGHT
1	I	I
2	II	II
3	IV	II
4	IV	IV
5	I	II
6	II	II
7	IV	III
8	II	II
9	II	II
0	II	II

RIGHT ⟷ LEFT	SPACING BETWEEN NUMERALS (mm)
I ⟷ I I ⟷ II II ⟷ I	19
I II ⟷ III IV III IV ⟷ I II II ⟷ II	14
III IV ⟷ III IV OPPOSITE	10
III IV ⟷ III IV PARALLEL	0

STANDARD NUMERALS

10-CENTIMETER HIGH