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, } [illegible], Colonel.

The 5th March 1951. } Military Secretary to the President

56-57

Journal of the Indian Roads congress

1956 - 57

INDIAN ROADS CONGRESS

ESTABLISHED : 1934 REGISTERED : 1937

OFFICE-BEARERS AND COUNCIL 1956-57

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3. C. J. Fielder
4. E. A. Nadirshah
5. L. R. Patel
6. N. Durrani
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10. C. M. Bennett
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16. A. C. Malhotra
17. Kishori Lal
18. U. J. Bhatt
19. K. K. Kartha
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24. G. D. Joglekar
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PERSONNEL OF THE TECHNICAL COMMITTEES AND SUBCOMMITTEES

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(*iii*) *Specifications & Costing Division*

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Chief Surveyor of Works, E-in-C's Br.
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H. S. Batliwala

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A representative of C. W. & P. C.
W X Mascarenhas
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A representative of Central Board of Irrigation & Power
Dr. K. L. Rao

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K. C. Sood — M. V. Joglekar
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S. N. Gupta — N. N. Majumdar
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Chief Engineer (Highways), Madras
Chief Engineer, Buildings & Roads, Punjab
A representative of Railways Testing & Research Centre, Ministry of Railways, Lucknow
A representative of Central Standards Office for Railways
A representative of the Poona Irrigation Research Station
A representative each from Lucknow, Madras, Karnal & Hyderabad Research Stations
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A representative of Soil Laboratory, West Bengal
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J. M. Trehan ... *Member-Secretary*

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A representative of C. B. I & P. — R. N. Dogra
S. N. Gupta — T. M. Menon
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Goverdhan Lal
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8. *MANUFACTURE OF ROAD MAKING MACHINERY AND MECHANIZATION SUBCOMMITTEE*

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L. B. Patel — A representative of E-in-C.
W. A. Griffiths — N. Durrani
C. M. Bennett — P. N. Srivastava
H. P. Sinha

9. *ORGANIZATION COMMITTEE*

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K. K. Nambiar — U. J. Bhatt
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10. *ROAD OPERATION COST COMMITTEE*

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Jagdish Prasad — T. S. Santhanam
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D. P. R. Cassad
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11. *LOCAL COMMITTEE OF THE TEST TRACK DIVISION TO SUPERVISE DELHI - MATHURA ROAD TESTS*

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Consulting Engineer (Road Development)
A representative of C.P.W.D.
J. M. Trehan

12. *CEMENT CONCRETE ROAD SURFACING SUBCOMMITTEE*

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K. K. Nambiar
C. L. N. Iyengar
M. S. Bisht
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C. S. Anantapadmanabhan ... *Member-Secretary*

The Consulting Engineer to the Govt. of India (Road Development) is an ex-officio Member of all Committees and Subcommittees.

INDIAN ROADS CONGRESS

NOTICES AND ANNOUNCEMENTS

CONTRIBUTION OF PAPERS TO THE INDIAN ROADS CONGRESS

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

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

2. **The complete Paper should be sent 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 time to study Papers well in advance of the date of the Session, it is hoped that Members will make an effort to send their contributions **as early as possible.**

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

3. The younger engineers are particularly requested to contribute Papers which, if required, will be suitably edited by the Indian Roads Congress Office before publication.

4. **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.

5. 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.

6. Rules and Regulations for the contribution of Technical Papers and the award of Medals are published on pages 9 to 16.

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

1. The following Medals are available for award:

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

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

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

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

2. PROCEDURE FOR CONTRIBUTION OF TECHNICAL PAPERS

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

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

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

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

3. RULES FOR THE CONTRIBUTION OF TECHNICAL PAPERS

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

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

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

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

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

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

4. SCRIPT

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

a. **All Contributions should be Drafted in the third Person.**

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

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

5. ILLUSTRATIONS

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

a. Line drawings.

b. Half-tones.

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

Line drawings are generally classified into two categories :

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

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

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

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

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

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

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

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

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

6. BRIDGE DETAILS

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

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

b. A dimensioned cross-section of the roadway;

c. The loading on which the design is based;

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

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

7. UNITS

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

STANDARD UNITS OF WEIGHT AND MEASURE

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

8. FORMULAE

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

9. ABBREVIATIONS

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

10. ITALICS AND EMPHASIS

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

11. PARAGRAPHING

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

I.

II.

1.

2.

a.

b.

(i)

(ii)

12. ACKNOWLEDGMENTS

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

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

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

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

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

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

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

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

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

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

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

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

IV. LIBRARY

1. Information and books on loan from the libraries of the Consulting Engineer (Roads) 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 Consulting Engineer (Road Developments') Office, but back issues of certain Journals may be sent on loan for a limited period.

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

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

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

6. **Instructions** :

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

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

7. **General** :

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

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

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

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

V. NEW ADMISSIONS

LIST OF MEMBERS OF THE INDIAN ROADS CONGRESS ENROLLED FROM 1-10-1956 TO 31-12-1956.

Serial No.	*Roll No.*	*Name*	*Address*
1	2165	P. S. Sanghera	Sub-Divisional Officer, P. W. D., B & R, **Kandaghat** (Simla Hills)
2	2166	T. D. Desai	Executive Engineer, Navsari & Dangs Division, **Navsari.**
3	2167	Mohd. Zayaullah-Hamidi	Engineer Assistant, Tirhut Pharmacy, Kalyani, **Muzaffarpur** (Bihar).
4	2168	Dr. H. C. Visveswaraya	Engineer, The Concrete Association of India, Cement House, Queen's Road, **Bombay-1.**
5	2169	A. P. Singh	Sub-Divisional Officer, P. W. D., Darbhanga Sub-Division No. I, **P. O. Laheriasrai** (Darbhanga-Bihar).
6	2170	S. S. Singh	Assistant Engineer, P.W.D., R. & B., Madhipura (South), Madhipura, **Dist. Saharsa** (Bihar).
7	2171	Lieut. Avtar Singh	74 Field Coy., C/o. 56 A. P. O., **New Delhi.**
8	2172	Panna Lal Sinha	Assistant Engineer, P. W. D., B/R. I/c. Samastipur Sub-Division, P.O. **Samastipur,** Dist. Darbhanga (Bihar).
9	2173	M. V. Rao	Junior Engineer, P. W. D., B. & R., Raipur Circle, **Raipur** (M.P.)
10	2174	H. U. Bijlani	Assistant Engineer (Consultant), Ministry of Transport, Jamnagar House, Shahjahan Road, **New Delhi-2.**
11	2175	P. D. Kataria	Construction Engineer's Office, Assam Oil Co., Ltd., **Digboi** (Assam).
12	2176	Olin Wesley Mintzer	Professor of Highway Engineering, Punjab Engineering College, **Chandigarh** (Punjab).
13	2177	Major M. L. Malhotra	Executive Engineer, Sunam Roads Division, **Patiala** (Punjab).
14	2178	T. Venkatacharyulu	16, D. S. Barracks, University of Roorkee, **Roorkee.**
15	2179	D. C. Gupta	Assistant Engineer & Technical Assistant to the Executive Engineer, Left Canal Division, **Kotah** (Rajasthan).
16	2180	H. R. Premaratne	Deputy Director, P. W. D., 12, McCarthy Road, **Colombo-7** (Ceylon).
17	2181	H. Raghavendra Rao	Technical Assistant to the Chief Engineer, (Highways), Andhra Pradesh, **Kurnool.**

VI. DEATHS

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

Serial No.	*Roll No.*	*Name*	*Address*
1	283	Sohan Lal	Executive Engineer (Retired), New Dak Bungalow Road, **Patna** (Bihar).
2	341	S. E. Akhtar	Executive Engineer, **P.W.D. Gaya** (Bihar).
3	1193	S. M. Kar	District Engineer, **Hamirpur** (U.P.)

VII. ADDRESSES WANTED

Communications addressed to the following Members have been received back undelivered. Members who are able to give information regarding the present address of any of them are requested to forward particulars to the Secretary, Indian Roads Congress.

Serial No.	*Roll No.*	*Name.*	*Address as last known.*
1	652	Guruswamy, S.	No. 8, Appaswami Koil Street, P. O. Mylapore, Madras.
2	1403	Khosla, B. R.	Garrison Engineer, No. 5, Cannaught Place, M. E. S., Dehra-Dun. (U. P.)
3	183	Nawab Ahsan Yar-Jung Bahadur	Afsar Manzil, Jubilee Hill, Hyderabad-Deccan.
4	417	Parang, Hari Ram	Retired Executive Engineer, Lal Chowk, Qila Gujar Singh, Lahore.
5	1743	Prasada Rao, A.M.V.	Managing Director, Ramprasad Ltd., 5, Sri C. V. Raman Road, Alwarpet, Madras.
6	1354	Srivastava, G. N.	C/o. Shri Chowdhury Nawal Kishore-Prasad, Advocate, Mohalla Pirmohani, Patna. (Bihar).

VIII. REMOVALS FROM ROLLS OF THE SOCIETY

List of Members of the Indian Roads Congress whose names have been struck off from the rolls in default of payment of arrears of subscription.

Serial No.	*Roll No.*	*Name of Member*
1	139	Murari Lal
2	156	Romesh Chandra
3	297	A. J. Khan
4	310	B. S. Puri
5	319	J. N. Das Gupta
6	364	M. A. R. Iyengar
7	365	M. L. Saberi Yousaff
8	402	A. C. Pujari
9	336	H. S. Bansor
10	451	Nilkanth Rath
11	452	J. M. Ajwani
12	616	Brij Narayan
13	632	Indramanikar
14	648	R. G. Perry
15	682	C. K. Sreekantiah
16	130	J. M. Todd
17	721	F. C. Acherjee
18	778	K. P. Laha
19	894	P. J. Hanly
20	905	H. G. Cocksedge
21	984	K. M. Mathew
22	1037	K. R. M. Nair
23	1076	M. A. Jabbar
24	1133	S. K. Banerjee
25	1204	A. C. Mathur
26	1229	R. P. Sawhney
27	1257	N. K. Sinha
28	1359	H. Seetharamiah

Serial No.	*Roll No.*	*Name of Member*
29	1361	Lakshmi Narayan
30	1369	D. N. Ganguli
31	1408	L. N. Kabiraj
32	1428	K. Govindarajulu
33	1437	K. V. Ahmed Koya
34	1460	K. Narasimha Rao
35	1465	K. Subba Rao
36	1472	A. R. Mastoor
37	1527	M. L. Champhekar
38	1563	A. C. Jayaraj

Paper No. 189

"LOAD TESTING OF THE PRESTRESSED CONCRETE BRIDGE ACROSS BHIMA RIVER AT TAKLI ON SHOLAPUR-BIJAPUR ROAD"

By

S. V. NATU, B.E.

CONTENTS

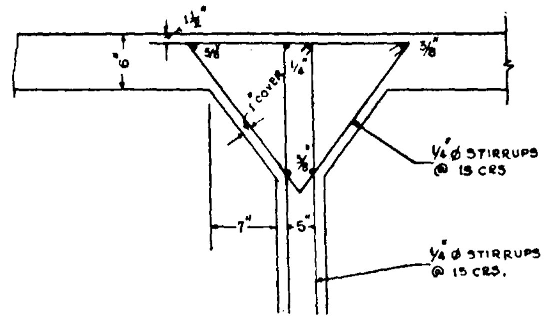
SYNOPSIS

The Paper describes the load testing of the prestressed concrete bridge across Bhima river near Takli on Sholapur-Bijapur road, 18 miles to the south of Sholapur. The bridge has 7 spans each of 153 ft 3 in. centre to centre of piers. In two of the seven spans the extensions of the prestressing tendons obtained during the stressing were considerably less than the calculated ones due to the sheaths having got wrapped round the cables. In order to get the designed prestress at the critical section, tendons of these two spans were restressed after a lapse of time.

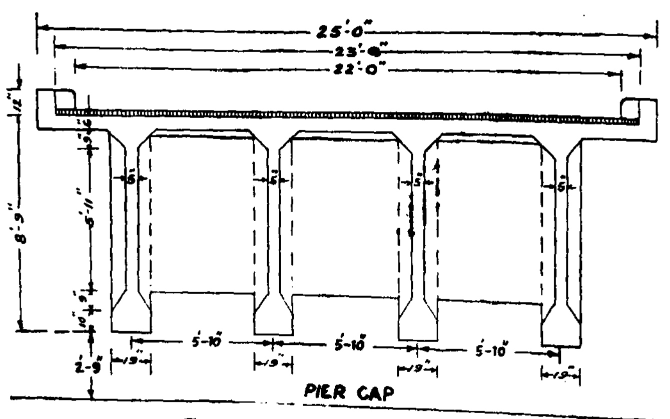
These two spans and one out of the rest were also tested for cracking load. Contrary to what was expected, testing revealed that the load carrying capacity of the beams in the two spans, in which the extensions obtained originally were less than necessary, was considerably deficient. The Paper also deals with the strengthening of the girders in the two spans.

1. INTRODUCTION

1.1. The bridge across Bhima river on the Sholapur-Bijapur road, a State Highway, at about 18 miles south of Sholapur town, has seven spans each of 153 ft 3 in. centre to centre of piers. The superstructure consists of 4 prestressed concrete beams to a span.



Details at the junction of web and flange of prestressed concrete T-beam.



Cross section through deck.
Fig. 1.

2. SUPERSTRUCTURE OF THE BRIDGE

2.1. General Description of the Superstructure

The superstructure consists of cast in-situ prestressed concrete T-beams simply supported on cast steel single roller and rocker bearings. R.C.C. slabs form the flanges of the prestressed concrete girders. Details of reinforcements at the junction of the web and flange of the prestressed concrete T-beams are shown in Fig. 1. The prestressed concrete beams are spaced 5 ft 10 in. centre to centre. The clear carriageway between R.C.C. guard stones is 22 ft and the distance between railings 23 ft 6 in. The overall width of the deck is 25 ft. The girders of the end spans are 150 ft and those of the intermediate spans 146 ft 9 in. centre to centre of bearings. The clear space on piers between the ends of girders is 4 ft which

is about the requirement for accommodating the stressing jack. The 3 ft 2 in. thick rib of the inverted R.C.C. T-beam forming the pier cap occupies this space after stressing is completed. A section through the deck is shown in Fig. 1.

7-in. thick R.C.C. diaphragms are spaced about 16 ft centres. These are reinforced with 4 numbers of $\frac{3}{4}$ in. diameter rods at top and 4 numbers of $\frac{1}{2}$ in. diameter rods at bottom. Two pronged stirrups, $\frac{3}{8}$ in. in diameter, are used at 10 in. centres. The diaphragms do not support the deck.

2.2. Materials and Design Stresses

Concrete for the end blocks is roughly 1 : 1 : 2 mix. The portions of the prestressed girder other than the end blocks, diaphragms and the deck slab are 1 : $1\frac{1}{2}$: 3 mix. The concrete is weigh batched and has a minimum strength of 5,000 lb per sq. in. at 28 days.

High tensile steel wires of 0.2 in. diameter with an ultimate tensile strength of not less than 100 – 110 tons per sq. in., and a working stress of not less than 66 tons per sq. in. were used.

2.3. Design criteria

Stresses are not to exceed –

Initial stress in steel	tons per sq. in.	66.7
Compressive stress in concrete in service	lb per sq. in.	1500
Tensile stress in concrete in service	lb per sq. in.	0

2.4. Casting of Girders

The prestressing was done on the Freyssinet system with cables having 12 numbers of 0.2 in. wires. The four girders of the end span had 25 cables each. Each of the girders of the intermediate spans had 24 cables each. The cable consisting of wires arranged round the central helix had a diameter of 1.02 in. The inside diameter of the metal sheathing was to be about eighth of an inch more than the diameter of the cable to enable the cable being inserted into the sheathing easily and to ensure that the cable was free to move inside the sheathing. Unfortunately, the metal sheathing of No. 32 gauge black iron sheets got wrapped round the cables rather tightly in spans 6 and 7 which were the first ones to be cast towards the end of the working season of 1954–55.

The concrete of the girders was compacted by internal immersion type vibrators and by external vibrators worked by electricity. The serious consequences of the defectively sheathed cables were obvious when they were stressed. The actual extensions of the tendons were far too short of the calculated ones for the permissible end stress.

This defect was removed while concreting the girders of span 1, which was taken up next, first in the season of 1955–56. In the absence of a sheath making machine, sheaths were pre-formed to the required diameter by hand using a pipe. In addition, to be on the safe side, two extra cables were provided in each of the four girders. Should the number of cables provided in the design fail to obtain the design prestress due to excessive friction between the cable and the sheathing, these extra cables could be stressed to make up the deficiency. Depending on the results obtained in this span, however, the extra cables were to be omitted in the succeeding spans. Happily, the contingency of using these extra cables in span No. 1 did not arise and they were pulled out and reclaimed for use elsewhere.

2.5. Calculated Extensions

The required extensions were calculated by adopting STUP's formula for frictional losses, which is as under:

Total frictional loss =

0.1 per cent of the end stress in cable per foot length of cable for the wobble effect,

plus

0.4 per cent of the end stress in cable per degree of curvature for curvature effect.

For a central stress of 66.7 tons per sq. in., allowing for losses due to successive prestressing of the cables which produced elastic shortening of the beam, the required extension calculated on the equivalent length of cable for a uniform stress as at end worked out to about 10.2 in. for end spans and 9.95 in. for the intermediate spans at an end stress of about 75 tons per sq. in.

2.6. Stressing

Stressing was done from both ends applying the stress equally. From the stressing of a few cables of spans 6 and 7, it was clear that there was excessive friction and at the permissible end stress, the extensions, obtained were considerably short of the calculated ones. To get an idea of the amount of friction, the cable was pulled from one end by the jack keeping the other end free. At the free end, movement of cable could be noticed at a pressure of about 40 tons per sq. in. (a total pull of about 15 tons). In the worst case, the pressure required to effect movement was as high as 50 tons per sq. in. With the precautions described in para. 2.4, stressing of the wires for the girders in span 1 gave satisfactory extensions. The average results obtained are as under:

Span No.	End of stress tons per sq. in.	Extensions obtained inches	Required extension inches	Deficiency per cent
6	76.0	7.054	9.95	29
7	74.9	8.877	10.20	13
1	76.1	10.388	10.20	nil

Deficient extension meant deficient prestress at the critical section and corresponding reduced moment of resistance. There were two methods of dealing with the problem. One was to strengthen the girders to make up the deficiency. The other was to restress the cables after a lapse of time to allow most of the losses due to creep and shrinkage of concrete to take place.

3. RESTRESSING

3.1. It was decided to try restressing and spans 6 and 7 were restressed after a lapse of about 9 and 10 months respectively after stressing. Inspection of a few inner cones removed from the girders revealed that they were damaged by wear due to original use and subsequent release. Their re-use might cause trouble by way of deferred slip of the wires after blockage.

3.2. Release of the stressed cables was accomplished as described below. The cables were stressed from one end until some movement took place. The movement indicated release of the inner cone on the jacked end. The jack was then removed and the inner cone, which was now fairly loose, was chiselled out. The jack was then applied at the other end and the inner cone on that end could be pulled out along with the wires without chiselling. Measurement of the distance between the file marks made on the cable at either end at a distance of 24 in. from the face of the beam during the initial stressing for the purpose of measuring extensions showed that, on release, the cables did not attain their original length. The total length measured from the face of the beam to the file mark should have been 48 in. if the cable was completely free from any stress. In most cases, the length measured more by several inches. This seeming non-return of the cable to the original length after being freed of end anchorages appeared to be due to locked-up or residual extension in the cable consequent on reversed friction preventing its return to the original unstressed condition coming into play on release of the end pulls. A part of it was, no doubt, attributable to the plastic set of the steel wires which was estimated to be about 0.4 inch. It is to be concluded that the locked-up extension over and above the plastic set indicated the extent of the stressed state and could be allocated towards the total extension of the cable by adding it to the restressed extension. Efforts were made to completely free the cable of any stress by pulling it 3 to 4 times alternately from either end by the jack. Only 3 to 4 cables were released at a time and restressed so that the effect of successive prestressing could be eliminated.

3.3. At the time of restressing, out of the total of 15 per cent losses due to shrinkage, creep of concrete, etc., 10 to 12 per cent could be assumed as having taken place. On account of the excessive friction encountered during the initial stressing it could be assumed that all the friction was due to wobble effect and it was uniform all along the length of the cable. For span 6 with an average length of cable of 150 ft, the extension required during restressing then worked out as given below:

	Tons per sq. in.
Central stress as per design	66.7
Deduct for losses due to shrinkage etc. having taken place (12 per cent approximately)	8.0
Central stress required at the time of restressing	58.7
Permissible stress at end	80.0
Average stress along the cable assuming uniform friction $\frac{(80.0+58.7)}{2}$ =	69.4

$$\text{Extension required} = \frac{69.4 \times 2240}{29 \times 10^6} \times 150 \times 12 = 9.6 \text{ in.}$$

If with an effective end stress of about 80 tons per sq. in. an extension of about 9.6 in. could be had, it could be taken that the desired prestress at the centre had been obtained.

Extension required for span 7 with an average cable length of 155.5 ft worked out to 9.9 inches. The results of restressing of the girders in spans 6 and 7 are summarised below:

Span No.	End stress tons per sq. in.	Average locked-up extension inches	Average extension obtained in restressing inches	Total extension inches
6	79.52	2.062	7.826	9.888
7	77.73	2.769	8.235	11.004

Thus, the extensions obtained seemed satisfactory after making an allowance of about 0.4 in. for the plastic set in steel. With a view to reducing friction between cables and ducts, in span 6 the ducts were filled with soap water. A few ducts were filled with a solution of one part of Dromus B, a lubricant, in 20 parts of water. This treatment, however, did not help as was evident from the results of restressing some of the cables where no lubricant was used. The average results in both the cases were the same. Apparently, there was no reason why friction should substantially reduce after a lapse of time. At best, overcoming a substantial part of the losses in stress due to creep and shrinkage only could be expected. Therefore, correctness of assuming the so-called locked-up extension as contributing to the stress was doubtful. Testing the girders for cracking load alone could establish the value of prestress at the central section. So, it was decided to load test the two spans.

During the restressing 8 wires in span 6 and 26 in span 7 got broken.

4. LOAD TESTING

4.1. To produce the desired bending moments, a uniformly distributed central load of sand bags, occupying a length of 20 ft and the full width of the carriageway of 22 ft, was applied, Photo 1. The surfacing and railings, which contributed to the dead weight, had not been done at the time of test. So, a dead load producing the same bending moment at the centre of the span as the surfacing and the railings was placed before the start of the tests. The test load was applied in increments of 10 tons uniformly spread over the loading area. At the end of each increment the deflection of each of the four girders was measured and load deflection curves were plotted for each of the girders.

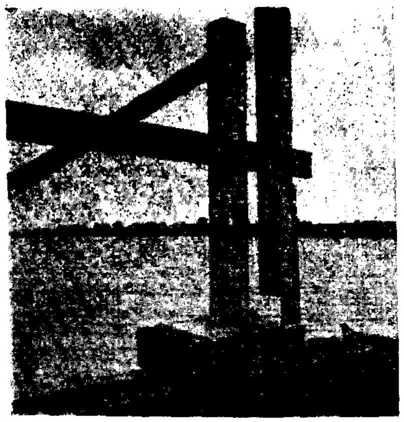


Photo 4.

Bottom of the vertical post showing the fulcrum and the registering arm.

Photo 5.

Dial gauges under the vertical post for measuring deflections.

Photo 6.

The bridge nearing completion.

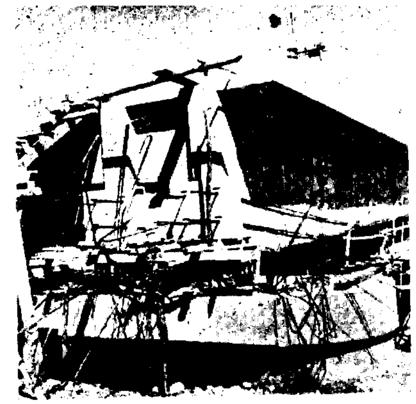


Photo 7.

R. C. C. end blocks for the strengthening cables.

Photo 8.

A close-up of the R. C. C. end block.

Photo 9.

A view of the strengthening cables and suspenders.

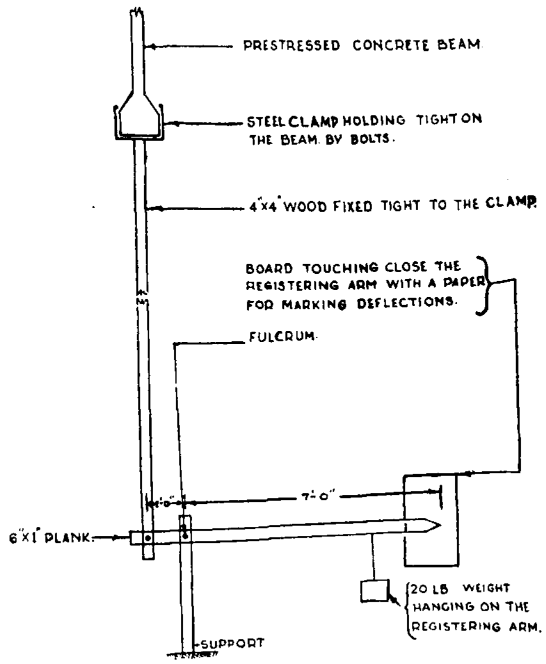


Fig. 2. **Device for measuring deflections.**

4.2. Device for Measuring Deflections

The deflections were measured by a locally manufactured device. The principle of the lever was used for magnifying the deflection. This device could read to an accuracy of 1/336th of an inch which was all that was needed. The deflections registered by the device were compared with those recorded by few dial gauges, and good agreement was found between the two. The measuring device is shown in Fig. 2 and Photos 2 to 5. A 20 lb weight was hung on the registering arm to eliminate the effect of loose bolted connections, if any, at the fulcrum and the vertical member. Also, the weight ensured that the vertical member was always pressed against the beam whether it moved up or down, and so the recorded deflections were not affected. One deflection recorder was installed under each of the four beams of a span. At about the ground level a platform was constructed to rest on wooden piles driven hard. There was also a platform at the bottom of the beams for observers to keep a watch on the development of visible cracks.

Photo 1.

Loading with sand bags.

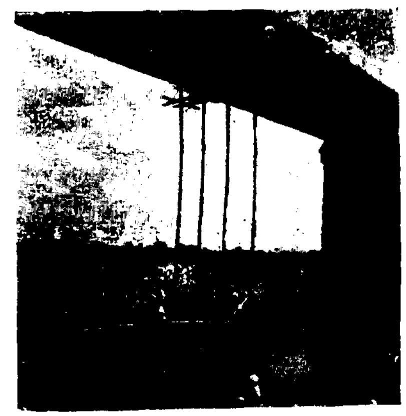


Photo 2.

Arrangements for measuring deflections.

Photo 3.

Close view of the iron clamp at bottom of the girder and the vertical wooden post.

Photo 10.

Arrangement for supporting the cable at the bend. Looking towards the end block.

Photo 11.

The stressing jack.

Photo 12.

A view of the completed bridge. The last two spans that needed strengthening are those on the left.

4.3. The test load was increased in 10-ton increments until visible cracks appeared in the bottom fibre of the beams, and then in 5-ton increments until the cracks extended some distance up the girders. Simultaneously the load deflection curves were being plotted and departure from linearity of the curves watched. The girders were watched at their centres for appearance of visible cracks. Tendons of the girders in spans 6 and 7 were not grouted before the test, whereas cables of span 1 were grouted. Ducts of girders in span 6 were filled with soap solution for lubricating the cables. Appearance of visible cracks in these girders was preceded by sweating which slowly developed into droplets. Between sweating and cracking there was an increment of 5 to 10 tons in the test load. The appearance of cracks fairly coincided with the point on the load deflection curves where it sharply deviated from linearity.

4.4. Second Application of the Test Load

After removal of the test load, fully in span 7 and to about 0.4 of the cracking load in spans 6 and 1, a second loading was applied in the same manner as the first one. In the second loading, as the tensile resistance of the concrete that had to be overcome before cracks could appear was not there, the cracks previously formed reopened at a lesser load than during the first loading. The second loading thus determined the load necessary to reduce the bottom fibre stress due to prestressing to zero. From this, the amount of prestress necessary at the bottom fibre for full dead load could be assessed by simple calculations. Comparison of this value with the design value of the prestress would indicate the measure of the success of prestressing.

4.5. Analysis of the Tests

The results of load tests and the comparison of the actual and the design prestresses are given in Table 1, page 45. Load deflection curves for average values of the four girders for span 6 are shown in Fig. 3. The curves for spans 7 and 1 are more or less similar. Magnified deflections (7 times), obtained on the registering board at the deflectometer for the upstream inner beam of span 7, are shown in Fig. 4. It will be seen from Fig. 4 that there was an overnight increase in the deflection because of contraction during the night due to fall of temperature. During unloading, in the afternoon recess of about an hour and a half, there was a small reduction in the deflection due to rise of temperature and consequent expansion. During unloading, there was again an overnight increase in the deflection. These increases and decreases in the deflections had been distributed uniformly

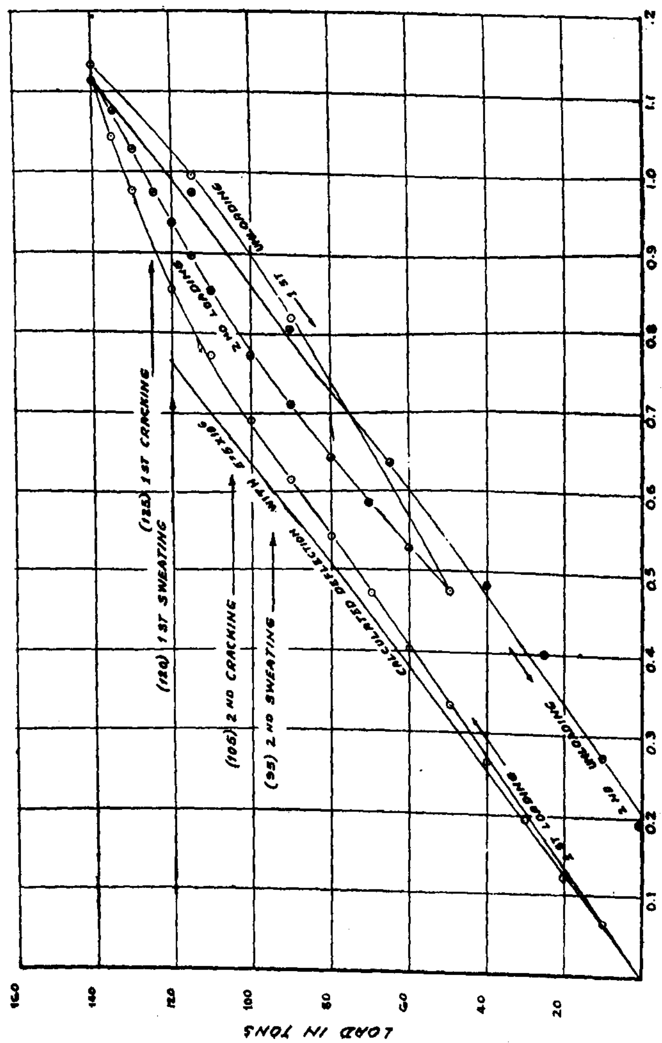


Fig. 3. Load deflection curve—span 6.

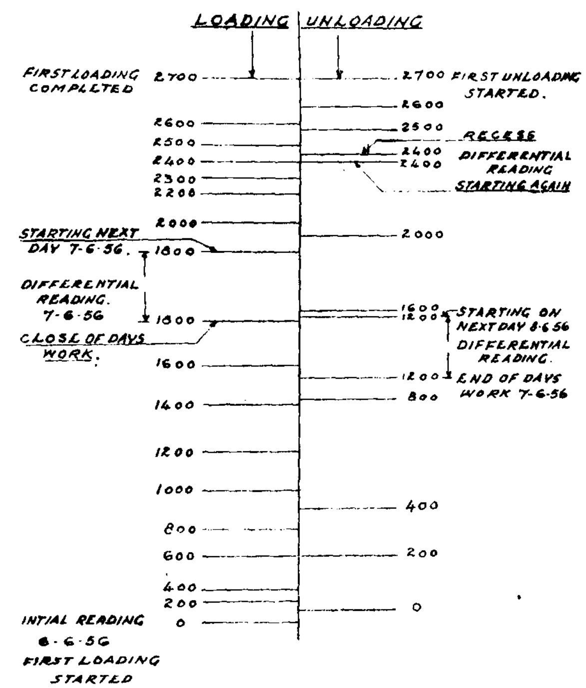


Fig. 4. **Deflections magnified 7 times during the first loading of upstream inner girder, span 7.**

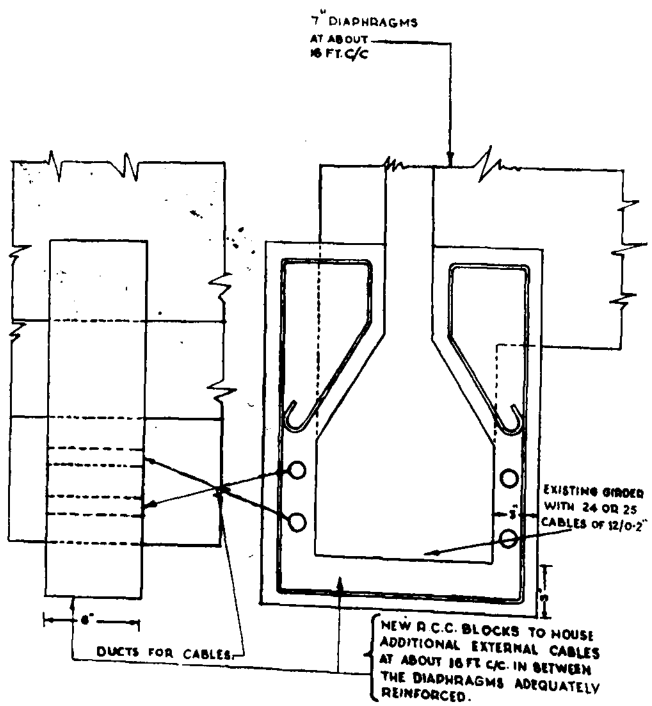
in the next few readings for the purpose of plotting, so as to have the curves continuous. This is the main reason why the load deflection curves in Fig. 3 are curved.

The graph of the first loading, Fig. 3, would have been near straight right up to sweating load, had there been no distribution of the overnight increase in deflection at 50 ton load in the next six readings. The common point of deflection at the end of the first unloading and beginning of the second loading is also not correct but is assumed, since the second loading was commenced on the next day. Fig. 3 also gives the calculated graph of deflection with value of $E = 5 \times 10^6$ lb per sq. in. The deflection according to calculations was 0.765 in. at a load of 120 tons, while the recorded deflection was 0.857 in., a difference of 0.092 in. Against this, the overnight increase in deflection of the girder at 50 ton load was 0.089 inch.

In Table 1, items 10, 11 and 12, the numbers of extra cables required for strengthening, taking into account the symmetry of stressing and the additional dead load involved, are given.

5. STRENGTHENING

5.1. The extra cables for strengthening were to be provided outside the bottom bulbs of the girders. These could be held in position by either of the arrangements shown in Fig. 5 and 6. The alternative shown in Fig. 6 was adopted. It was decided to



ELEVATION
Diaphragm not shown.

SECTION

Fig. 5. Arrangement for holding strengthening cables in position.

stress the strengthening cables from one end only since there was no problem of friction. Cables of span 7 were stressed at the abutment end and those of span 6 were stressed on the second pier from the abutment. Where the cables changed direction there would be a vertical force which would have to be taken care of. This force, however, was not excessive as the cables deflected by 6 deg. once only. Reinforcement shown in Fig. 5 was meant to take care of the tension that might be created by the vertical force.

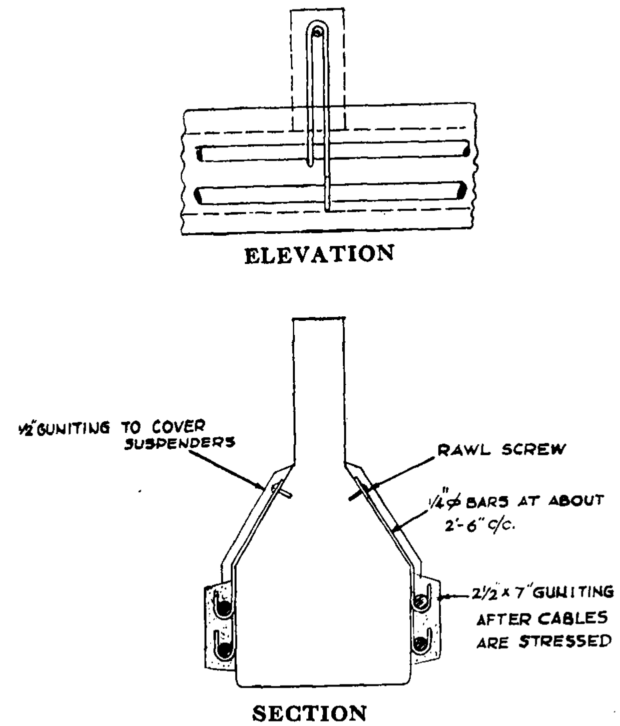
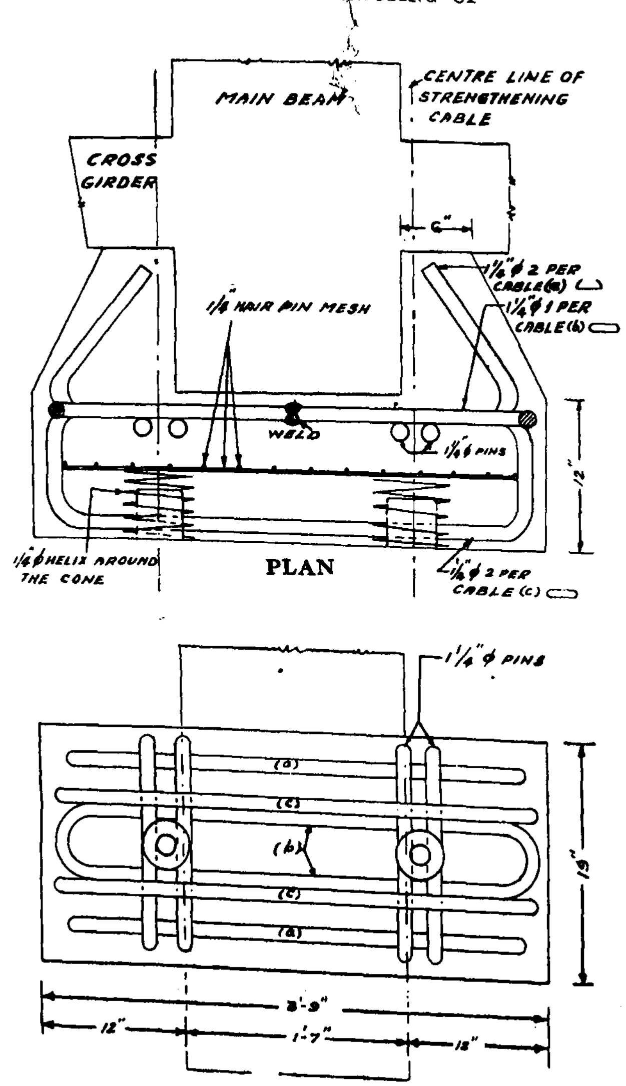


Fig. 6. Alternative arrangement for holding strengthening cables in position.

5.2. The end block for one pair of cables is shown in Fig. 7. In span 6, there will be one pair of cables each for the outer girders. For the inner girders of span 6, and for all the four girders of span 7, there will be two pairs of cables each. The end block for girders which are to be strengthened with four cables, is approximately double than shown in Fig. 7 in elevation. Each cable exerts a force of about 27 tons on the end block and the shear intensity for the block of dimensions shown works out to about 370 lb per sq. in. on bja area. No doubt, a part of the cable force is transferred directly to the beam causing no shear in the block. However, the whole of the shear is provided for by adequate reinforcement (acting as rivets). The bearing pressure on the concrete by the reinforcement is limited to 1,500 lb per sq. in. With a safe shear stress of 5 tons per sq. in. in steel, four $1\frac{1}{4}$ in. diameter bars are adequate. The bars are required to project straight beyond the face of beam by 8 in. to develop the required bearing strength with the above mentioned stress in concrete. Bars marked '*c*' in the Fig .7 take care of



ELEVATION

Fig. 7. **Details of end block.**

the possible bending moment that is likely to be developed in the block. The C. G. of the extra cables is about 3 ft below the neutral axis of the beam at a section near the end. The direct and bending stresses developed by the extra cables do not produce any tension in the top fibre of the beam.

5.3. Each pair of cables is stressed simultaneously and the order of stressing by girders will also be symmetrical, so as to minimise eccentric stressing.

After stressing, the cables were grouted with neat cement using "Lissapol N" and a water cement ratio of 0.45. In certain cases where obstruction to the passage of the grout was encountered, grouting was done by applying the grout nozzle at intermediate points in the cable by tapping the sheath. The cables were finally covered over by gunniting. The beam face was slightly roughened by chiselling so that the gunniting may stick to the beam satisfactorily.

During execution, the anchor blocks were set at a slight outward angle so as to get enough space for the full run of the jack during stressing. The average extension required was about 10 inches. The jack could thus run just clear by the side of the opposite beam end. Where space difficulty was still encountered due to inadequate splay of the anchor blocks stressing was done in two stages, i. e., tensioning was first done half-way then the jack was released and reset at the start of the run and further tensioning done until the desired extension was obtained. Though a pair of cables was stressed at a time for symmetrical prestressing, stressing of one cable at one time had not any adverse effect on the end block as was evidenced by complete absence of any hair crack or movement of the end block. This lends support to the assumption that part of the cable force is transferred directly to the beam face. Photos 7 to 11 show various stages of strengthening.

Under live load, there is a change in stress of over 900 lb per sq. in. in the bottom fibre and the gunniting would crack when the live load is passing. The cracks would, however, close again. As the cables are sheathed ones, there will not be material danger of corrosion of high tensile wires.

6. CONCLUSIONS

6.1. In the first stressing, the deficiency in extensions in spans 6 and 7 was 29 and 13 per cent respectively. Disregarding the locked-up extensions the actual restressed extensions were respectively 18 and 17 per cent short of requirement. The test loading showed the deficiency of prestress in these two spans to be 10 and 15 per cent respectively. Restressing had thus improved the performance of span 6 considerably but not that of span 7. From the results obtained with girders in span 6 it had to be concluded that not only a part of the friction had been overcome in restressing but also a part of the measured locked-up extension. This indicated the stressed state of the cables when unlocked by removal of inner cones at either end. There had been fewer breakages of wires, 8, during restressing as well as during the initial stressing in span 6, and the above mentioned deficiencies did not need much of a correction on this account.

Performance of span 7 showed no improvement. There had been comparatively more breakages of wires, 26, in span 7. Cables in which wires had snapped during the initial stressing had not been restressed. These breakages affected the observed results. The measured average locked-up extension for span 7 was quite large but had not influenced the performance of the span at all. Therefore the measurements for the locked-up extension might not be correct perhaps due to the adoption of a different datum for its measurement from the one adopted while making file marks on the cables during the initial stressing. The latter datum might have been either the face of the beam or the face of the support ring which again was likely to vary in length with the jack used for stressing.

TABLE 1

Results of Load Tests

		Span 6	Span 7	Span 1
1. **Details of Span.**				
Length centre to centre of bearings	Ft – in.	146 – 9	150 – 0	150 – 0
2. **Design—Service Stresses**				
(a) Top fibre in concrete	Lb per sq. in.			
(i) Under dead load only	,,	+ 728	+ 763	+ 763
(ii) Under dead and live load	,,	+1448	+1504	+1504
(b) Bottom fibre in concrete				
(i) Under dead load only	,,	+1131	+1165	+1165
(ii) Under dead and live load	,,	+ 186	+ 191	+ 191
3. **Moments and Loads.**				
(a) Design live load moment in span	Ft – lb (in millions)	9.37	9.71	9.71
(b) Uniformly distributed central load over an area of 22 by 20 ft on the span which produces a moment equal to that given in (a) above	Tons	123	124	124
(c) Moment required to reduce the bottom fibre stress to zero	Ft – lb (in millions)	11.22	11.58	11.58
(d) Uniformly distributed central load which produces a moment equal to that mentioned in (c) above	Tons	147	148	148
(e) Moment required to reduce the bottom fibre stress to – 180 lb per sq. in., the assumed tensile strength of the concrete at bottom fibre	Ft – lb (in millions)	13.02	13.38	13.38
(f) Uniformly distributed central load which produces a moment equal to that mentioned in (e) above	Tons	170	171	171
4. **Stresses in the bottom fibre due to:**	Lb per sq. in.			
(a) Full dead load	,,	–2070	–2167	–2167
(b) Full dead and live load	,,	–3015	–3141	–3141
(c) Live load alone	,,	– 945	– 974	– 974

TABLE 1 (*contd.*)

		Span 6	Span 7	Span 1
4. Stresses in the bottom fibre due to	Lb per sq. in.			
(d) Due to prestressing (after 15 per cent assumed losses) so as to have a residual stress as per 2 b (ii) above under full dead and live load	,,	+3201	+3332	+3332
(e) (d) − (a) above	,,	+1131	+1165	+1165
(f) Due to prestressing (after 15 per cent assumed losses) so as to have a residual stress of +50 lb per sq. in. only under full dead and live load	,,	+3065	+3191	+3191
(g) (f) − (a) above	,,	+ 995	+1024	+1024
5. Results of first loading (average for 4 girders).				
(a) Load at sweating	Tons	120*	—	—
(b) Stress in bottom fibre (f_b†) at sweating load	Lb per sq. in.	−923	—	—
(c) Load at visible cracking	Tons	125*	101*	160*
(d) Stress in bottom fibre (f_b†) at cracking load	Lb per sq. in.	−963	−797	−1264
6. Results of second loading (average for 4 girders).				
(a) Load at sweating	Tons	95*	—	—
(b) Stress in bottom fibre at sweating load	Lb per sq. in.	−731	—	—

* For details see Table 2 on page 48

Properties of the combined 4 girders (all spans) $Zt = 90.5 \text{ ft}^3$

$Zb = 69.0 \text{ ft}^3$

Moments in million ft—lb.	Span 6	Spans 7 & 1
D. L.	20.58	21.55
L. L.	9.37	9.71
Total	29.95	31.26
Prestress Final	22.85	23.8
Prestress Final direct stress lb per sq. in.	+ 940	+ 901

$$\dagger f_b = M/Z$$

$$= \frac{(T \times 2240)/2 \times (146.75 - 10)/2}{69 \times 144}$$

$$= 7.693\ T\text{—lb per sq. in.}$$

Where T = Central load in tons

and Z = 69 ft^3 units,

TABLE 1 (*contd.*)

		Span 6	Span 7	Span 1
6. Results of second loading (average for 4 girders.)				
(c) Load at visible cracking	Tons	105*	76*	140*
(d) Stress in bottom fibre at visible cracking load	Lb per sq. in.	−803	−599	−1105
7. Average tensile strength of concrete at bottom fibre.				
5 (b) or (d) — 6 (b) or (d)	Lb per sq. in.	192	198	159
8. Prestress available at the bottom fibre at the centre of span on the day of testing.	Lb per.sq. in.			
(a) 4 (a) + 6 (b) or (d)	,,	+2801	+2766	+3272
(b) Deduct for losses at 2.5 per cent still to take place (out of 15 per cent)	,,	−70	−69	−82
(c) Available stress in service	,,	+2731	+2697	+3190
9. Deficiency in prestress.				
(a) For residual as per 2 (b) (ii) 4 (d) — 8 (c)	Lb per sq. in.	470	635	142
do. do.	per cent	14.6	19.0	4.3
(b) For residual of +50 lb per sq. in. 4 (f) — 8 (c)	Lb per sq. in.	334	495	1
do. do.	Per cent	10.4	15.0	0
10. Design number of cables.	Number	96	100	100
11. Additional cables required for 9 (b)above	Number	10	15	—
12. Cables actually provided talking into account the extra dead load involved in strengthening and considering symmetry.	Number	12	16	—

* For details see Table 2 on page 48.

TABLE 2

Details of Test Load

(Tons)

	Down-stream outer girder	Down-stream inner girder	Up-stream inner girder	Up-stream outer girder	Average
Span 6—First Loading					
At sweating	115	115	120	130	120
At visible cracking	120	125	125	130	125
Span 6—Second Loading					
At sweating	90	95	100	100	95
At visible cracking	100	105	105	110	105
Span 7—First Loading					
At visible cracking	102	96	111	96	101
Span 7—Second Loading					
At visible cracking	79	73	79	73	76
Span 1—First Loading					
at visible cracking	163	157	157	163	160
Span 1—Second Loading					
at visible cracking	145	135	135	144	140

Paper No. 190

"STUDY OF A CLAY BANK FAILURE ON AN INDIAN RAILWAY"

By

U. G. K. RAO

CONTENTS

SYNOPSIS

A clay embankment, about 35 ft high, on the Central Railway was giving trouble from the time of its construction. The trouble in the earlier stages appeared to be due to inadequate slopes. Also, some of the remedial measures then adopted, though not justified when judged by principles of soil mechanics might have reduced the strength of the embankment. More recently, the bank seemed to have been affected by moisture movement both from the top and from the ground below.

In 1955, detailed investigations were carried out. The Paper gives the history of the embankment from the date of its construction, the observations made during the investigation, and the recommendations made for remedying the defects.

1. INTRODUCTION

1.1. A double line clay bank constructed in 1912 between Ajni and Nagpur on the Bhusaval-Nagpur section of the Central Railway was reported to be giving trouble to maintenance engineers almost since the date of construction. Before 1912, both the up and down lines in this section were on an old embankment. With the development of traffic and laying out an elaborate yard, a new embankment was constructed using locally available clayey material. It would be seen from the cross-sections, Plate I, that there were, at the moment, three embankments, one on the right-the old embankment which carried both the up and down lines before 1912, the second one for the receiving and departure lines in the Ajni Yard and the embankment which was constructed in 1912. The embankment which was reported to be giving considerable trouble was the third one carrying the up main line and the up and down engine line. It was reported in 1955 that the embankment below the up main line slided. The damage to the embankment did not extend to the up and down engine line and, therefore, the traffic was being worked on this line for the duration of the breakdown. The embankment was later made up by coal ashes, and a dry rubble toe wall as indicated in the cross-section at the top of the Plan, Plate II was also provided. For investigating the cause of the slip, a detailed examination of the area and the soil conditions was undertaken and recommendations made for remedying the defects.

2. HISTORY

2.1. Practically from 1912, the year of construction, almost every monsoon, trouble was being experienced in the form of settlements and minor slides, etc. But detailed information of what took place after 1940 only was available for study. In the monsoon of 1940, a portion of this embankment between miles 518/16-18 (Telegraph Posts) slipped and the embankment underneath the up-line slumped as a result. It was reported that considerable water was found to ooze out from the slope of the embankment at a point almost mid-way along the slope. It was, therefore, felt that the best solution would be to provide drainage for the water which was collecting in the body of the embankment. For this purpose, riser pits and leaky pipes were provided and a pucca masonry toe wall constructed in certain places. The cross-section to which these riser pits were constructed is shown in Plate III. The work concerned was completed in 1941.

2.2. Again, in 1942 while the portion repaired in 1941 appeared to be satisfactory, the adjacent bank sections, viz., at miles 518/14–16 and 18–20 (Telegraph Posts) behaved in an identical manner during the monsoon. As the bank provided with the riser pit drains had appeared to stand up and the zone of trouble seemed to have moved further, it was decided to treat these milages also on the same lines.

2.3. Since then the report from the maintenance engineers was that during the monsoon when the formation was very well saturated, cracks developed on the top and progressively extended to a point between the up main line and the up and down engine lines. With the development of these cracks and moisture coming out from the slope, the embankment soil

slumped. A 9-in. water-main connecting Nagpur with Ajni running on the embankment almost invariably got distorted as a result of the slumping and burst, thus causing more and more havoc in addition to presenting difficulties to the restoration operations. In 1942 it was also felt by the engineers that the surface water coming down between the embankment was percolating into the up main line embankment. It was decided to catch up this water and drain it away and the following works were carried out in 1943.

(a) Construct riser pits and provide leaky pipes in the portion of the embankment between miles 518/14–16 and 518/18–20;

(b) Lay out a rubble drain as per sketch shown in the plan, Plate II, in the elevated portion between the ash pit siding and the up and down engine lines;

(c) Shift the 9 in. diameter water main On account of certain difficulties in obtaining the piping material, all other items of work, other than the shifting of the pipe line, were completed in the summer of 1945. Thereafter this embankment suffered minor settlements, etc., and appeared to behave satisfactorily until September 1955.

2.4. In 1955, almost the same trouble repeated. A 2-in. wide crack about 3 ft deep developed for a length of 100 ft on the up cess of the up embankment between miles 518/15–16, and this crack went down the slope of the embankment at the end of the 100-ft portion at an angle of nearly 60° right down to the toe. As was usually done on the Railways, this crack was made up by unloading coal ashes and traffic was passed at a restricted speed. But soon after another crack developed in the middle of the formation under the up track and this resulted in the movement of the water main running to Ajni. The bank definitely showed signs of slumping and the engineers first proposed to support the main as, if it burst, it would cause more damage to the entire embankment. In spite of all efforts, further deterioration of the bank prevented the supporting of the water main and when it burst the whole embankment from the middle of the up track formation started slipping and got washed away. Immediately, the engineers had to provide more reliable means of support for the pipe line and the entire slipped portion of the embankment was made up with coal ashes.

3. FIELD INVESTIGATIONS

3.1. The results of an 'on-spot' survey of riser pits provided in 1941 and 1945, made in October 1955 are given in Appendix 1. The survey showed that the riser pits and leaky pipes had all got disturbed and all of them were not functioning satisfactorily.

3.2. The rubble drain constructed between the ash pit siding and the up and down engine lines was completely distorted and was out of line and level. It was reported that the condition of the drain had been so from almost the time it was constructed. It was, therefore, clear that this drain was not functioning. On the other hand, it appeared that water might have been flowing through this drain into the body of the embankment instead of being led away.

3.3. Local examination indicated that the problem was not connected with surface flow on the top of the embankment and it might be more due to sub-surface flow below the bank which caused substantial hydrostatic head in the body of the embankment. Therefore, a further examination was made of the ground conditions by excavating a number of pits between the embankments and taking out undisturbed soil samples. Such pits are indicated in the plan in Plate II. The directions from which water oozed into these pits from the side in October 1955 are also indicated by arrows. In addition, water levels in the various pits and shallow wells constructed in the basin were recorded.

3.4. With a view to study conditions of the soil in the embankment samples were taken with a soil auger from the slope of the up embankment in the portion where no damage had taken place and in the portion between the engine line and the up main line. In addition, water levels in the various pits and wells in the locality were recorded in the summer of 1956. The water levels in the monsoon in the wells are also shown in Plate II. Details of the above mentioned observations and test results of soil samples extracted from different locations are indicated in Plates I and II. Important observations made during these investigations are recorded below:

(i) As could be seen from the sample of soil collected from the up slope of the undisturbed portion of the embankment trial pit No. 1, material used for constructing the old embankment had as much as about 50 per cent clay. This clay, however, did not appear to be a typical black cotton soil having high shrinkage and swelling properties. In addition to possessing low friction it was impermeable, and

thus had more harmful effect on the stability of the bank. Almost the entire portion of the embankment which had been giving trouble during the last many years, had been replaced by coal ashes leaving the original soil only under the engine line which appeared to be substantially clayey. Soil augering in the body of the embankment, however, proved that almost everywhere resistance was met with from rubble, boulders, etc., beyond a depth of about 8 ft below the formation level. Without a detailed information about the work done on the embankment from 1912 to 1939, it could only be assumed that material of all kinds, viz., ashes, ballast, boulders, etc., might have been used for making up the embankment whenever it showed signs of failure.

(ii) Soil samples obtained from the slopes of the embankment etc., particularly on the upside, appeared to be mixed with coal ashes and other coarse grained material as they had an appreciable value for the angle of internal friction and were also more permeable than the original clay which was met with nearer the toe.

(iii) The ground levels indicated that the three banks had all been put up in a basin shaped formation and this basin had a small ridge beneath the down track formation sloping away towards the up track. This ridge encouraged flow of both surface and subterranean moisture away from it in either direction.

(iv) It was understood that considerable accumulations of water had taken place on either side of the embankment when the slide occurred and the levels recorded in October 1955 in different places also went to prove that water in the pits between the reception line formation and the up line formation was about 8 to 10 ft above that on the upside of the failed embankment. It was also possible that this difference in water levels might have been much more before the bank actually failed.

(v) When excavations were made in the basin on which these embankments had been formed, it was seen that except for a top layer of 2 to 3 ft of cohesive soil, granular material which was found to be a water-bearing stratum existed at lower levels. During the monsoon, apart from the movement of moisture that might be taking place in this layer in the initial stages, due to rise of level of water into the cohesive soil, capillary rise of moisture was obviously taking place into the body of the embankment. This capillary moisture combined with moisture from the top must be collecting in the pockets of non-homogeneous material used for the making up of embankment, and causing severe hydrostatic pressures resulting in the failure of the embankment. It was seen that, during the summer, the water level all over was more than 6 to 7 ft below the ground level and the bank, therefore, gave no trouble.

3.5. It has been already pointed out that the riser pits which were provided to drain the body of the embankment had discontinued to function. This was not surprising as almost every year the bank was sinking and getting disturbed. This resulted in shifting the pipes as well as the drains. It seems that the coal ashes used in the embankment might have choked the rubble packing, thus preventing drainage after the first or the second monsoon. The ground levels on the upside of the up embankment as well as the sill level of the 8 ft arch culvert proved that during the monsoon the surface drainage and the subsoil drainage were defective. Therefore, during the monsoon, when the ground water level rose considerably on either side of the embankment, a good hydraulic gradient might be existing across the embankment. Before the failure, it was likely that the gradient might be almost horizontal for the portion where the original clayey soil under the up and down engine lines existed. This would result in a steeper hydraulic gradient in the portion below the up main line. The lowest line assumed is shown dotted in the cross-section at Ch. 1400 in Plate I.

4. RECOMMENDATIONS

4.1. It would, therefore, appear that the failure of the embankment was essentially due to defective surface and subsoil drainage. Existence of a non-cohesive layer about 3 to 4 ft below the ground level was an advantage for subsoil drainage. If this layer was satisfactorily drained into the nearby arch culvert, considerable relief should be possible. Surface drainage on the upside of the embankment would have to be adequately provided for so that no water accumulated during the monsoon on the upside. The recommended remedial measures are given in the following paras :

4.2. A deep drain be provided to the section as shown in the plan in Plate II in the ground between the up line formation and the formation of the reception and departure lines. The bed level of the arch culvert and the drainage on the down stream end be adjusted to suit the outfall of this drain.

4.3. To eliminate the accumulation of moisture, particularly at the toe of the up slope which might, to some extent, have caused minor scours resulting in washing away of the embankment soil, the up area be satisfactorily drained.

4.4. The 9 in. diameter water pipe line be shifted from the bank to rest on independent supports.

4.5. By excavating below the engine line, a continuous homogeneous layer of coal ashes, edge to edge, eliminating clay or other non-homogeneous material, be provided up to about 6 ft below the formation level. Between the ballast and the ashes used in the embankment a layer of graded material be placed to serve as an inverted filter so that rain water might not percolate into the body of the embankment.

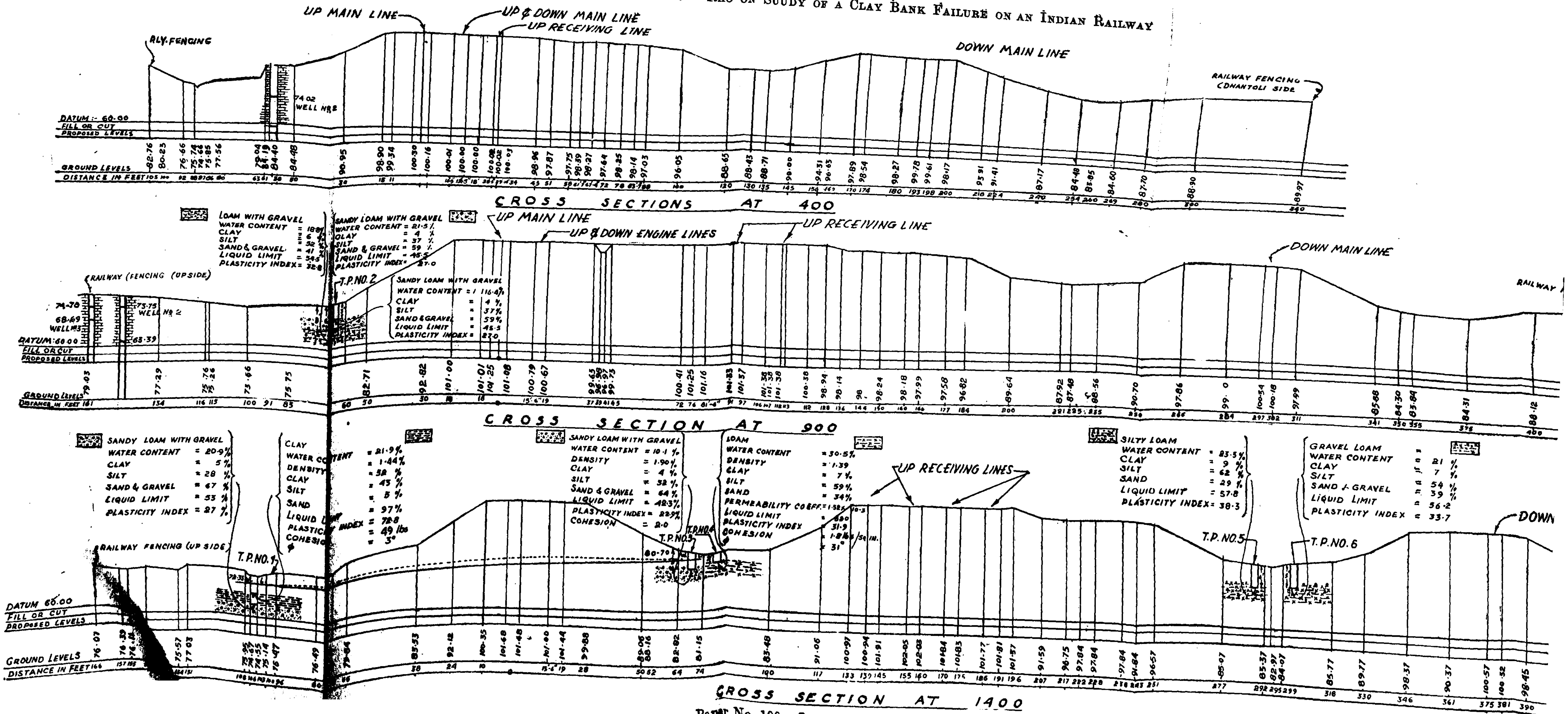
4.6. The up slope of the embankment be neatly dressed and a good rubble wall to hold it be provided, as had been done at chainage 300 and 500.

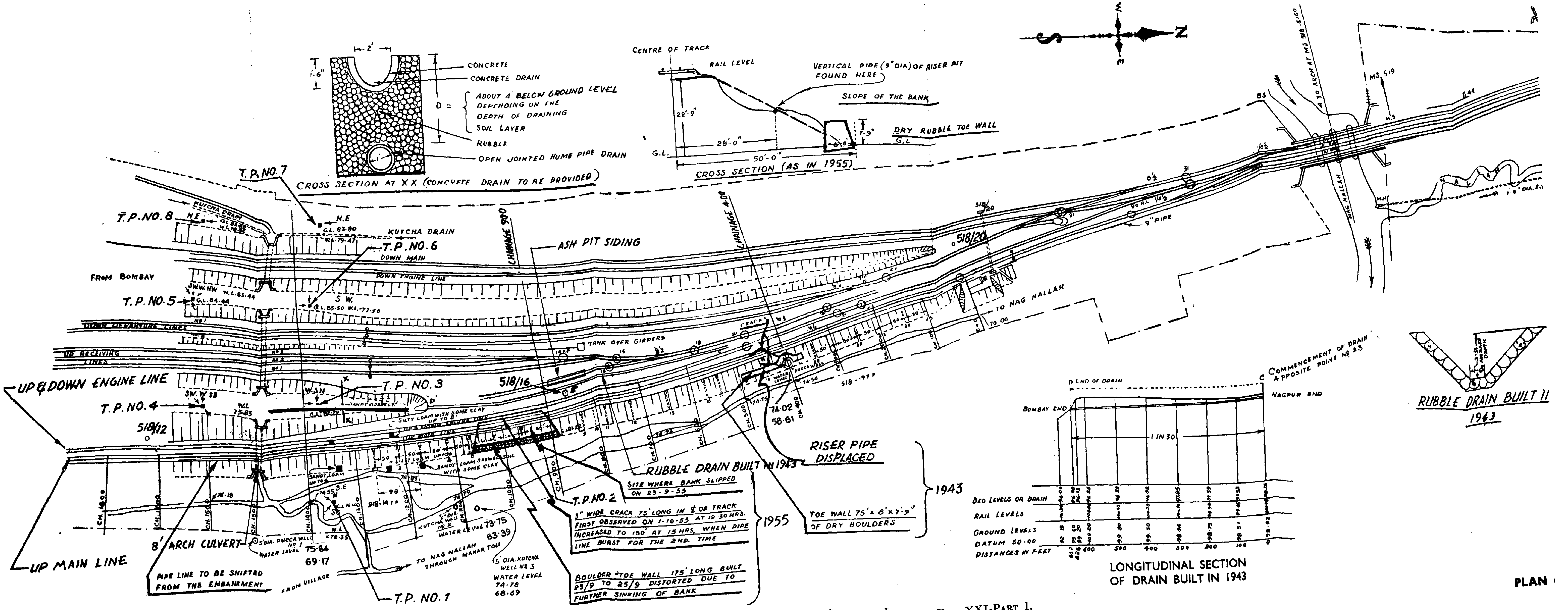
4.7. The drain between the up track formation and the marshalling lines be rebuilt and made ‘pucca’ so that moisture would not go through it. This is necessary because a defective drain might be harmful.

Appendix 1

RESULTS OF AN 'ON-SPOT' SURVEY OF RISER PITS

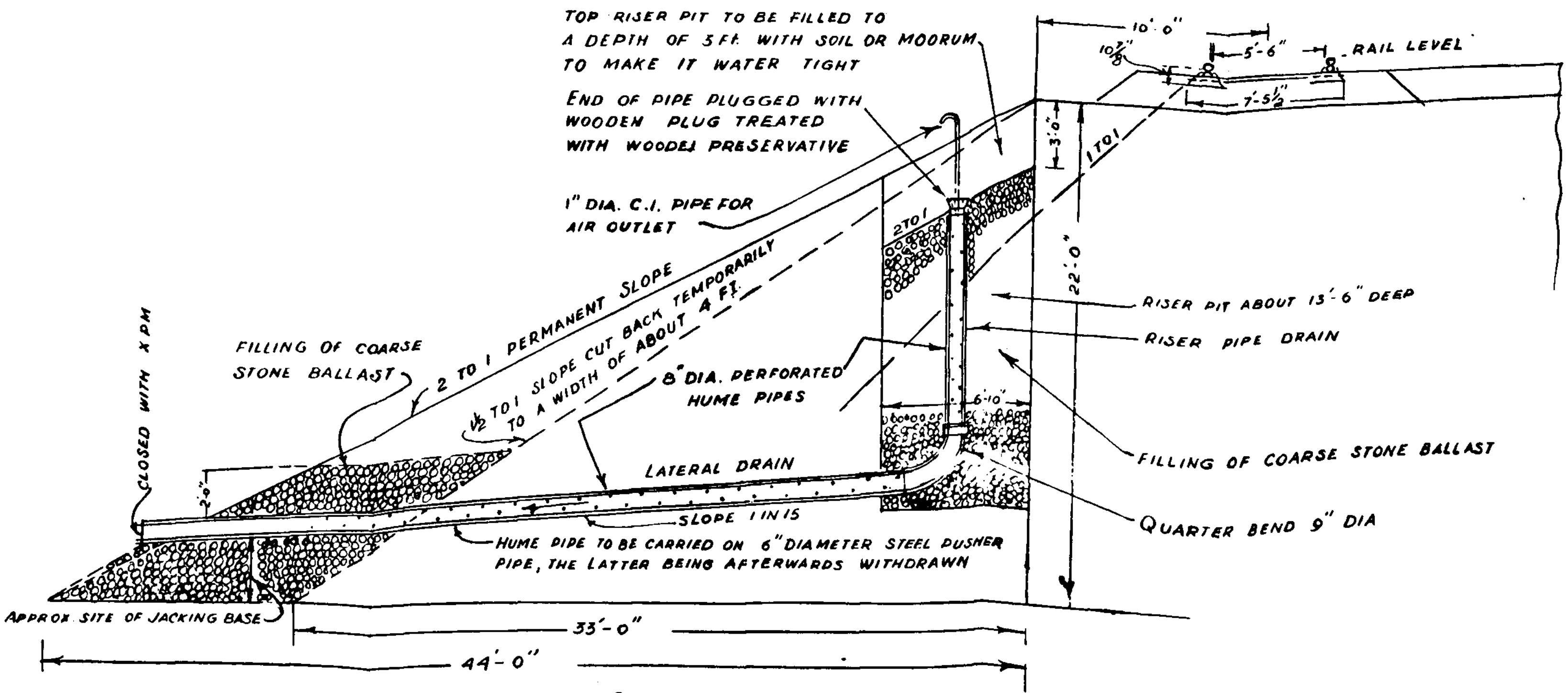
Riser Pit No.	
1	Miles 518/14, No trace.
2	Horizontal pipe visible. Vertical pipe not traceable.
3	Existing intact.
4	Not traceable.
5	Not traceable.
6	Vertical pipe displaced due to bank slipping on 1-10-1955 (Bombay-end portion of the slipped bank).
7	No trace though water was seen flowing freely.
8	Not traceable. (Nagpur-end of slipped bank on 1-10-55).
9	Not traceable. Water was seen flowing freely.
10	Horizontal pipe existing. Vertical pipe not traceable.
11	Not traceable. At the bottom of the bank, masonry bund wall provided. Water flowing freely.
12	Not traceable. Water flowing freely.
13	Horizontal pipe existing. Vertical pipe not traceable. Water is flowing freely.
14	Horizontal pipe existing. Vertical pipe not traceable. Water was flowing freely.
15	do. do.
16	do. do.
17	do. do.
18	Pipe not traceable. Masonry wall built at bottom of bank to prevent slipping.
19	Removed due to the well existing.
20	No trace of riser pit though water was seen flowing freely.
21	Existing with cork.
22	Existing without cork.
23	Existing without cork.
24	Existing with cork.
25	Existing with cork.


UP MAIN LINE
UP & DOWN MAIN LINE
UP RECEIVING LINE
RLY. FENCING
74.02 WELL NO. 2
DOWN MAIN LINE
RAILWAY FENCING CDHANTOLI SIDE
DATUM :- 60.00
FILL OR CUT
PROPOSED LEVELS
GROUND LEVELS
DISTANCE IN FEET
CROSS SECTIONS AT 400
LOAM WITH GRAVEL
SANDY LOAM WITH GRAVEL
UP MAIN LINE
UP & DOWN ENGINE LINES
UP RECEIVING LINE
DOWN MAIN LINE
RAILWAY (FENCING (UP SIDE)
T.P. NO. 2
WELL NO. 2
CROSS SECTION AT 900
SANDY LOAM WITH GRAVEL
CLAY
LOAM
UP RECEIVING LINES
SILTY LOAM
GRAVEL LOAM
RAILWAY FENCING (UP SIDE)
T.P. NO. 1
T.P. NO. 3
T.P. NO. 4
T.P. NO. 5
T.P. NO. 6
DATUM 60.00
FILL OR CUT
PROPOSED LEVELS
GROUND LEVELS
DISTANCE IN FEET
CROSS SECTION AT 1400



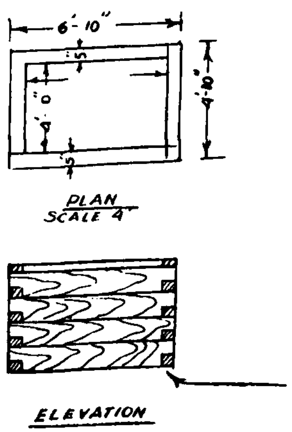
CONCRETE
CONCRETE DRAIN
ABOUT 4' BELOW GROUND LEVEL DEPENDING ON THE DEPTH OF DRAINING SOIL LAYER
RUBBLE
OPEN JOINTED HUME PIPE DRAIN
CROSS SECTION AT X X (CONCRETE DRAIN TO BE PROVIDED)
CENTRE OF TRACK
RAIL LEVEL
VERTICAL PIPE (9" DIA) OF RISER PIT FOUND HERE
SLOPE OF THE BANK
DRY RUBBLE TOE WALL
CROSS SECTION (AS IN 1955)
T.P. NO. 7
T.P. NO. 8
KUTCHA DRAIN
T.P. NO. 6
DOWN MAIN
FROM BOMBAY
DOWN ENGINE LINE
T.P. NO. 5
DOWN DEPARTURE LINES
UP RECEIVING LINES
UP & DOWN ENGINE LINE
T.P. NO. 3
T.P. NO. 4
518/12
518/16
518/20
CHAINAGE 900
CHAINAGE 400
ASH PIT SIDING
TANK OVER GIRDERS
TO NAG NALLAH
NAG NALLAH
RISER PIPE DISPLACED
RUBBLE DRAIN BUILT IN 1943
SITE WHERE BANK SLIPPED ON 23-9-55
T.P. NO. 2
3" WIDE CRACK 75' LONG IN ℄ OF TRACK FIRST OBSERVED ON 1-10-55 AT 12-30 HRS. INCREASED TO 150' AT 15 HRS. WHEN PIPE LINE BURST FOR THE 2ND. TIME
1955
TOE WALL 75' x 8' x 7'-9" OF DRY BOULDERS
1943
BOULDER TOE WALL 175' LONG BUILT 23/9 TO 25/9 DISTORTED DUE TO FURTHER SINKING OF BANK
T.P. NO. 1
UP MAIN LINE
8' ARCH CULVERT
PIPE LINE TO BE SHIFTED FROM THE EMBANKMENT
FROM VILLAGE
TO NAG NALLAH THROUGH MAHAR TOLI
5' DIA. KUTCHA WELL NO 3 WATER LEVEL 74.78 68.69
WATER LEVEL 73.75 63.39
75.84 69.17
74.02 58.61
END OF DRAIN
COMMENCEMENT OF DRAIN APPOSITE POINT NO 83
BOMBAY END
NAGPUR END
1 IN 30
BED LEVELS OF DRAIN
RAIL LEVELS
GROUND LEVELS
DATUM 50.00
DISTANCES IN FEET
LONGITUDINAL SECTION OF DRAIN BUILT IN 1943
RUBBLE DRAIN BUILT IN 1943
PLAN

PLATE III



CROSS SECTION

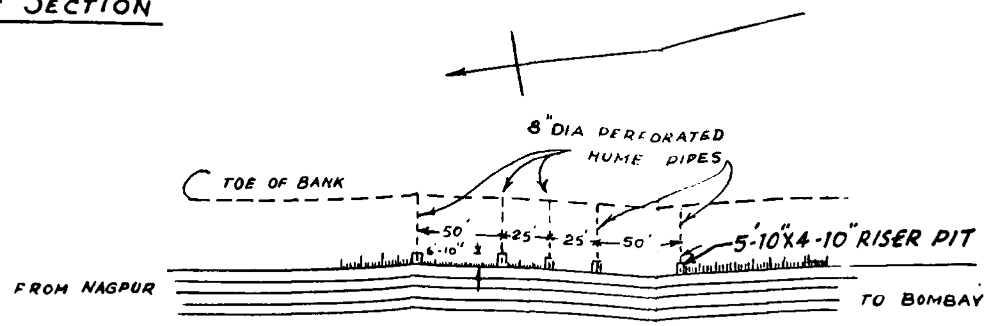
DETAIL OF SHORING FOR RISER PIT (MADE FROM OLD SLEEPERS)



Note :

These rings of shoring are to be put in position one beneath the other, as the excavation of riser pit proceeds, and are to be dismantled in a similar way. The ballast filling of riser pit keeping pace with the dismantling of shoring.

Shoring must extend down to top level of lateral drain.



SITE PLAN

DETAILS OF RISER PITS

Paper No. 191

"RESISTANCE TO ABRASION OF CONCRETE PAVEMENTS"

By

P. J. JAGUS AND M. R. CHATTERJI

CONTENTS

SYNOPSIS

The Paper emphasises the importance of considering the resistance to abrasion in the design of concrete mixes for rigid pavements and in the selection of materials and methods of construction. Poor resistance to abrasion is more serious in rigid pavements than in flexible surfaces.

It is considered that the standard tests in use for judging the wearing qualities of aggregates and the limiting values of abrasion resistance specified in the different specifications may not be suitable for conditions in India, where the principal damaging force is that of the steel-tyred bullock cart. So, a method of testing abrasion, called Pneumatic Test, was evolved at the Central Road Research Institute, Delhi. The test could be used for testing aggregates as well as cement mortars. The apparatus used in the test and the test procedure are described.

The wearing qualities of some road aggregates used in India determined by the Pneumatic Test are compared with the values obtained from other standard methods of tests. It is hoped that the pneumatic test will yield reliable results.

The effect of compressive strength, fineness modulus of sand used for the mortar, lapses in curing, use of various membrane curing compounds, and replacement of cement by surkhi on the resistance to abrasion of mortars has been studied and included in the Paper. It is found that coarser the sand that is used for the mortar the lesser the resistance to abrasion.

1. INTRODUCTION

1.1. General

It is important that considerations be given to abrasive resistance of pavements when selecting materials for construction and designing the mix and thickness of pavements, because of the presence of iron-tyred traffic using the roads.

1.2. Abrasive Wear of Pavements

Wear by traffic affects all types of wearing surfaces in a like manner but in varying degrees. Its effect on water-bound macadam and black-top construction cannot be considered as serious as it is for rigid pavements like concrete. Also, greater damage to non-rigid construction is through disruption due to impact, suction, and wheel drag of excessive traffic than by abrasion. While, in non-rigid pavements, the damage due to effects of traffic can be repaired through periodic maintenance, it is difficult to carry out such repairs for rigid pavements.

1.3. Wear by Abrasion and its Effects on Concrete Pavements

In concrete pavements, defects caused by lack of care in selection of materials, inadequate design and inadequate control in construction, did not lend themselves to easy correction. It was extremely difficult efficiently to set right damage to concrete due to abrasion except by resurfacing at a high cost. Patch repairs with asphaltic or cement concrete could be extremely unsightly and could hardly stand up to any reasonable traffic for long periods. Abrasion caused thinning out of designed sections, subjected the pavement to greater impact due to uneven riding surface, and weakened the subgrade due to rain water finding its way through cracks and open joints. All these resulted in a progressive failure of the pavement. Therefore, care in (a) selecting suitable aggregates, (b) designing the concrete mix adequately, and (c) exercising controls in construction, was essential. A concrete pavement had a longer life as compared to other types of construction, but deficiencies in its abrasion qualities could cause failure at sometime or other of its long life, specially because the surface neither needed nor got any surface maintenance. Even when design and control during construction were

up to the required standard, poor abrasive quality of the concrete alone could be sufficient to cause failure. Although construction of concrete pavements had not so far been extensive in India as compared to other types of roads, examples of severe damage through abrasion alone were not wanting. The nature and extent of such deterioration in some of the important roads are illustrated in the Photos 1 to 8.*

1.4. Need for the Study

However rapid the development in highway transportation be due to the five-year plans, the bullock cart would continue for a number of years in the foreseeable future. It would not be surprising if at an interim stage of industrial development of the country, greater use of roads than at present was made by bullock carts for movement of raw materials and finished goods. Also, iron-tyred traffic contributed, in a substantial measure, to wear of rigid pavements by abrasion. That being so, the need for paying greater attention to abrasion in the construction of concrete roads is imperative.

But this aspect had not received attention it deserved probably due to lack of extensive evidence of damage due to abrasion and the absence of such experiences in western countries. So an investigation of the problem was undertaken by the Central Road Research Institute.

2. METHOD OF TESTING ABRASION— THE PNEUMATIC TEST OF THE CENTRAL ROAD RESEARCH INSTITUTE

2.1. Approach to the Problem

Concrete was a heterogeneous material consisting of two principal ingredients, aggregates and mortar. Each one of these was important from point of abrasive qualities of the finished product. Whilst abrasion of the aggregate would be primarily governed by its own quality, that of the mortar would be governed by the design of the mix, nature of its compaction and finish, degree of control, and the quality of sand used.

2.2. Requirements of the Test

For direct or indirect measurement of abrasion resistance of aggregates, standard methods of test such as Deval attrition, Los Angeles rattler and Dorry disc tests, existed. But there

* With acknowledgments to the Concrete Association of India.

were no standard methods for testing cement mortars or concrete. The first phase of the study was, therefore, devoted to evolving a method of test for measuring abrasion resistance which would not only be applicable to coarse aggregates but also to cement mortars and concretes. The test evolved should be simple, not involve the use of heavy or expensive equipment, and consistently yield reliable and accurate results.

2.3. Survey of Existing Methods of Tests

Study of the methods[1 to 11*] in use for the measurement of wear resistance of concrete revealed that the investigator had invariably chosen a method of test used elsewhere or evolved one to suit his requirements and means. This led to the development of a number of methods based on different principles. Every one of the methods, however, had its own advantages, drawbacks and limitations. Existing methods might be classified into the following groups using :

(a) rattlers with 6 in. concrete cubes[1,2]

(b) rotating discs[3,4]

(c) wheels or casters revolving in a fixed orbit on test slabs[5]

(d) water jets with and without abrasives, and

(e) air jets with abrasives[6,7a8]

Although some of the above methods might have been considered satisfactory for giving an idea of the wear resistance, the degree of repeatability or consistency of the results obtained would be doubtful on account of one or more of the following factors :

(i) interference of abraded particles,

(ii) change in efficiency of the abrading medium from one test to another,

(iii) non-uniform distribution of the effort on the area tested, and

(iv) absence of simulated field conditions.

2.4. The Pneumatic Test – C. R. R. I. Method

After carrying out extensive tests extended over a period of one year, a simple cabinet was designed and a test

* Refers to the References on page 88,

designated Pneumatic Test was standardized for use in the investigation. A brief account of this first phase of the study, the description of the apparatus, and the standardized method are given in Appendix 1. Work with a view to improve the apparatus and the method for making them simpler, versatile and accurate was continuing. The principal advantages in the use of this method are :

(a) Coarse aggregates, mortars and concrete could be separately tested under identical conditions. The data so collected would be helpful in a balanced design of a concrete mix.

(b) The impingement of the medium used for abrasion at an angle very near to 90 degrees simulated the cutting action of bullock cart rims which were the principal destructive force.

(c) Inclination of the direction of the jet by 10 degrees to the vertical gave a shearing force similar to the drag of the wheel which caused dislodgment of the particles.

(d) The high velocity of particles used for producing abrasion simulated to a certain degree the effects of impact on pavement surface so conspicuously absent in some of the drag tests on concrete.

2.5. Action of the Steel Tyres

The action of the steel-tyred traffic on a road surface was two-fold :

(a) Steel rims cut vertically (i.e., knife edge action) in which the particles of coarse and fine aggregates exposed to the steel rims were crushed.

(b) The horizontal thrust of the wheels (both pneumatic and steel) accompanied by the impact caused dislodgment of particles of sand or coarse aggregates from the binding matrix.

2.6. Nature of Failure

Effect of the above mentioned action on coarse aggregates and mortar matrix would be failure either through crushing of softer grains in the coarse and fine aggregate or through failure of bond between the particles as a result of poor quality binding matrix in coarse aggregate or cement matrix in mortars. Although abrasion resistance of a finished surface had to be judged by its overall performance, it was governed by the hardness of particles comprising the aggregates or strength of binding matrix.

3. ABRASION RESISTANCE OF AGGREGATES

3.1. Tests on Some Road Stones

With a view to compare results, samples of major rock types used as road aggregates in India were tested using the pneumatic test and other standard tests.

The samples were supplied by the Chief Engineers of various State Public Works Departments, and included :

(1) Jalore granite (Rajasthan)
(2) Bengal gneiss (Chaibasa)
(3) Bundelkhand granite gneiss (Jhansi)
(4) Rajmahal trap (West Bengal)
(5) Bairenkoda quartzite (Cuddapah)
(6) Quartzite (Delhi)
(7) Limestone (Chittral)
(8) Deccan trap (Nagpur)
(9) Deccan trap (Bombay)
(10) Kushalgarh-limestone (Banswara)
(11) Muth quartzite (U. P.)
(12) Darjeeling gneiss (W. Bengal)
(13) Deccan trap (Poona)
(14) Limestone (Balasinor)
(15) Sandstone (Sagar)

3.2. The various test values of these stones are presented in Table 1, page 89, and Fig. 1. They are arranged in the descending order of the abrasion resistance obtained in the pneumatic test. The results indicate that the Deval wet attrition test bore closest similarity to the pneumatic test.

All the physical tests for abrasion of aggregates were hypothetical to a considerable degree and not much evidence existed of any definite correlation between the test values of aggregates and their field performances. On account of the presence of the steel-tyred bullock cart, the importance of evaluating these test values in terms of the field performance of the aggregates was great. Experiments at the Road Research Laboratory, England [15] showed that the co-relation between the abrasion and attrition test values of aggregates and their service performance was much better when the tests were carried out wet. Since the co-relation between the Pneumatic test and wet Deval attrition test was very high, the Pneumatic test could also be considered to be an equally good means of judging the service behaviour of the aggregates.

3.3 Analysis of Test Results

Table 2, page 90, gives the limiting abrasion test values specified by different authorities for use in various types of

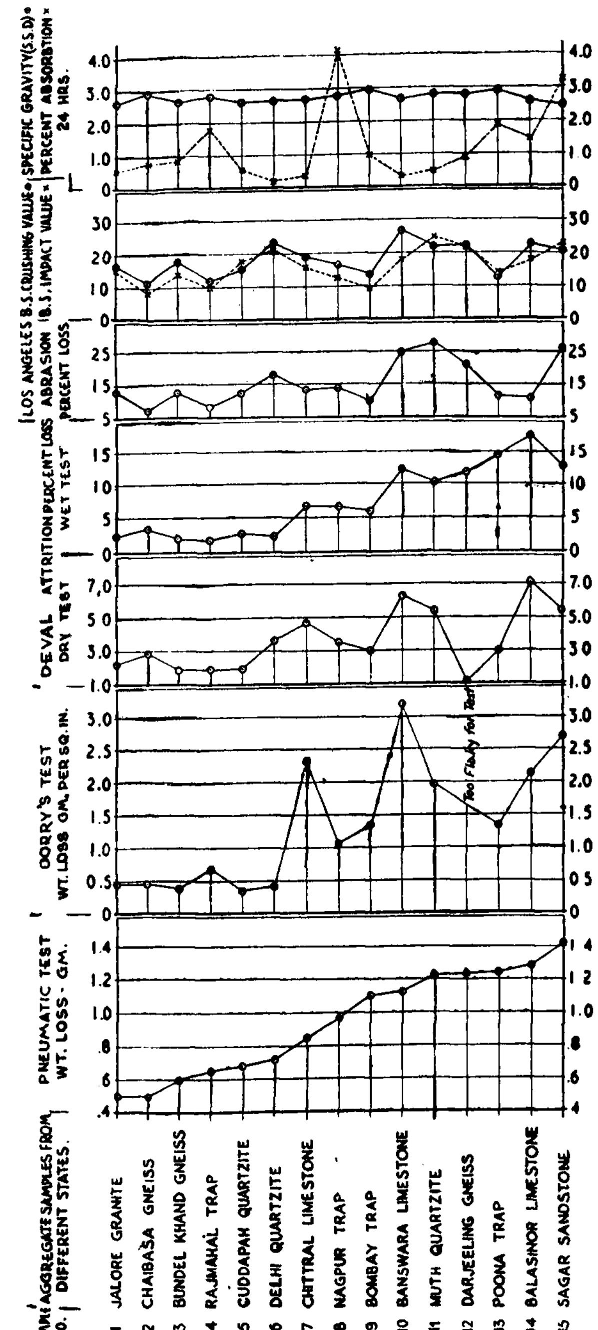


Fig. 1 Comparative physical test values of some Indian road stones.

road construction. It could be seen that all the aggregates tested satisfied the minimum requirements for use given in the various specifications. However, it was apparent even from visual examination that the aggregates bearing No. 11 and 15 (Muth quartzite and Sagar sandstone) would have poor wearing qualities under the bullock cart traffic. On the other hand, their respective Los Angeles values of 28 and 26 per cent were much below the specified limits and hence they could be considered to be good quality stones. Sample 13, Poona trap, the Los Angeles value of which was only 12 per cent (which was comparable with three other trap samples 4, 8 and 9) would be considered a very good quality stone. The loss in the pneumatic test, however, for all the three aggregate samples was significantly high. Sample 13 (Poona trap) could be said to have a poor field performance in abrasion as could be seen from the photographs of concrete surfaces on Poona-Ahmednagar and Bombay-Poona roads where similar trap aggregates from local sources were used.

It might, therefore, be assumed that certain characteristics associated with the resistance to abrasion of aggregates under steel rims were not brought out by the standard Los Angeles abrasion and Devals dry attrition tests. Therefore, judging the resistance to abrasion of aggregates by these tests for use under Indian conditions deserved a careful consideration. Also, whenever these test values were to form the basis for judging the quality of aggregate, the minimum values for Indian conditions should be far below those recommended by the A. A. S. H. O. or other specifications which were meant to cater only for pneumatic tyred traffic and not for severe crushing and shearing action of the bullock carts. It is hoped that the Pneumatic test might be a more reliable means of judging the resistance to wear of aggregates by iron rimmed bullock cart wheels.

4. ABRASION RESISTANCE AS A CRITERION FOR DESIGN OF CONCRETE MIXES

4.1. Compressive Strength related to Abrasion Value by Pneumatic Test

In a concrete mix it was the resistance of the mortar that governed abrasion. Therefore, using a good quality hard quartz sand, the variation of resistance to abrasion with the compressive strength of mortars was first studied. The fineness modulus and grading of sand were such as to give a dense mortar. Mortars of various strengths were obtained by changing the water-cement ratio. Cement : sand ratios were varied in order to get a uniform workability with different water-cement ratios. A flow of 35 per cent was adopted for all cases. In addition, one drier and one wetter consistency than the one giving 35 per cent flow was adopted for each cement : sand ratio to check the overlap of values. 2.78 in. cubes were cast and cured under water for 14 days at 86° ± 2° F. The relationship between the resistance to abrasion and the compressive strength of the mortar cubes tested is given in Fig. 2.

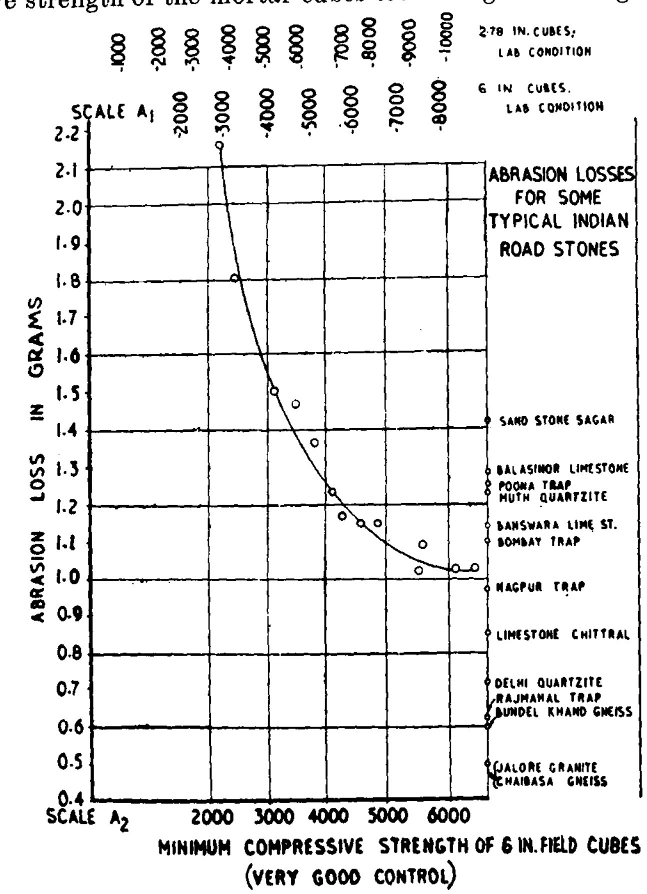


Fig. 2. Relationship between compressive strength and resistance to abrasion of cement mortars.

4.2. These data could be utilized for designing the mix proportions and water content of the concrete mixes. The compressive strength values shown in Fig. 2 were on mortar cubes cast and cured under laboratory conditions. For purposes of mix design, these values needed modification to allow for the effects of size of specimen and difference in the degrees of control in works and the laboratory. The effect of size of specimen could be taken into account by using data[12] obtained by research. In Fig. 2, the horizontal scale (A_1) at top gives the strengths of 6 in-cubes. These were obtained by applying a reduction coefficient of 0.82 on the values obtained with 2.78 in. cubes. Differential control between laboratory and works could be allowed for by multiplying these values again by a reduction coefficient depending upon the nature and degree of field control. The following values recommended in Road Note No. 4 of the Road Research Laboratory, U.K. could be adopted:

Very good control	0.75
Fair control	0.60
Poor control	0.40

Abscissa scale A_2 gives the further modified values of minimum field cube strengths based on very good field control.

Fig. 2 shows that the rate of decrease in the resistance to abrasion was high for compressive strengths below 4000 lb per sq. in. On the other hand, increasing the compressive strength beyond a certain optimum value would not give any appreciable increase in the resistance to abrasion. Probably, beyond the optimum, the increase of bond strength with increase of compressive strength was not proprotionately high.

In Fig. 2 the resistances to abrasion for some of the road stones tested have been shown. From the Figure it is clear that for concrete with good quality stone, like certain granites, the mortar would govern the abrasion resistance. So, attempts must be made to improve the quality of the mortar by, (1) selection of good quality sand, (2) using good grading, and (3) controlling the water content, compaction, etc. In such concrete, poor abrasion resistance of the mortar would lead to ravelling and gradual dislodgment of coarse aggregates from the matrix.

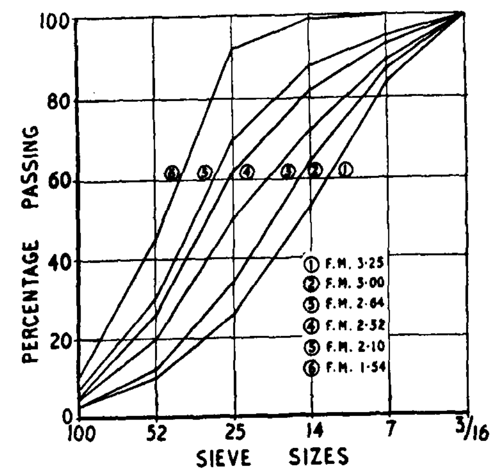
When poor quality aggregates were used, there were indications from other research works that increased cement content of the mortar helps in offsetting the reduced compressive strength to a very great degree. But, that the abrasive resistance of concrete was increased by increasing the quantity of cement, needed to be established. However, it could be said that when using softer aggregates, the overall performance of the wearing surface could be so improved that it functions better than a conventionally designed mix, by designing a mortar of very high resistance to abrasion and by so laying the concrete that the exposed surface presented a greater area of mortar than the coarse aggregates. In such cases, in addition to compressive strength, factors such as fineness and grading of sand, use of admixtures, special methods of compaction and finish, deserved consideration.

5. SAND FOR MORTAR

5.1. Further studies on abrasion resistance of mortars with different gradings of sand cement and water contents, workability, etc., were conducted.

5.2. **Effect of Fineness of Sand on Abrasion Resistance with varying Workability and Fixed Compressive Strength**

Mortar mixes were designed to give constant compressive strength using fixed water-cement ratio of 0.40 by weight and cement-sand ratio of 1 : 2. Six different sands of fineness modulii varying from 1.54 to 3.25 and gradings as shown in Fig. 3, conforming to the gradation recommended by the British Standard*, were used[14]. The same hard quartz sand employed in the earlier tests was used. The workability of the mix naturally varied and, therefore, the degree of compaction was adjusted to give maximum density.



Eig. 3. **Grading of sand.**

F. M. = Fineness Modulus.

Fig. 4 gives the effect of fineness of sand on the abrasion resistance of the mortar when cement-sand ratio was constant and the compressive strength and density nearly the same.

It could be seen that the density and compressive strength were very nearly constant whereas the abrasion resistance

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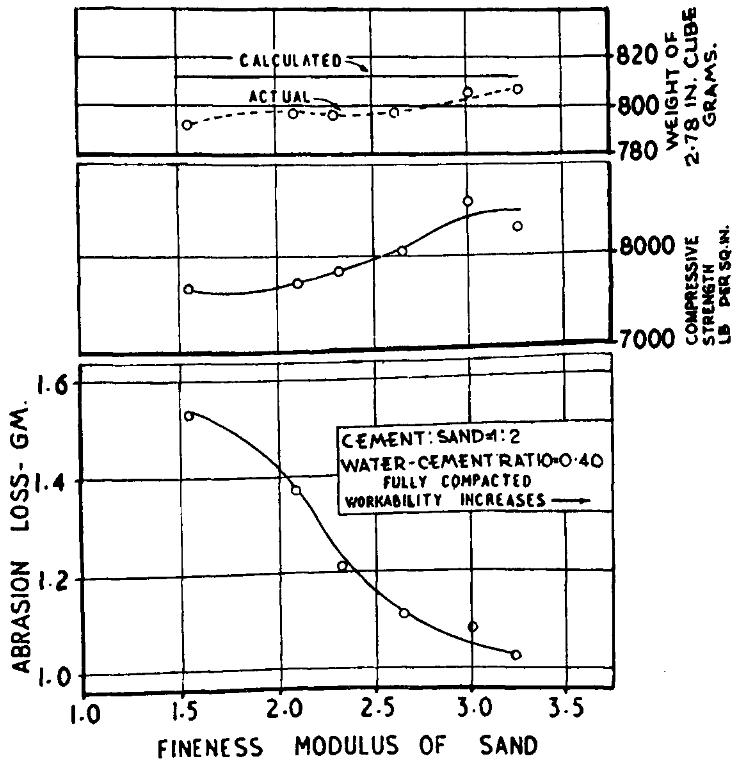

Fig. 4. **Effect of fineness of sand on resistance to abrasion of mortars with varying workability and fixed compressive strength.**

values changed significantly with the fineness modulus and grading of sand. For identical strengths, the abrasion resistance improved with increased fineness modulus of sand, provided the density of mortar was maintained constant. It was observed that the mix with coarsest sand required minimum of compaction. Concretes having such mortars would be ideally suited as they would allow a rapid, thorough and easy compaction of concretes, and provide a surface which would be durable under abrasion.

5.3. Effect of Fineness of Sand on Abrasion Resistance with fixed Workability and Compressive Strength

The principal factor governing the workability of concrete mixes for identical water contents was the surface area of the materials, namely, cement and aggregates. When cement and

Photo 1.
Great Western Trunk Road at mile 4/5 between Madras and Poonamalle.

Photo 2.
Great Western Trunk Road at mile 5/5 between Madras and Poonamalle.

Photo 7.
Haunches on Poona-Ahmednagar Road mile 2/5.

Photo 8.
Haunches on Poona-Ahmednagar Road mile 2/5.

water content were the same, the coarser sands, which had lesser surface area, yielded more workable mortars than finer sands which took up larger quantity of water for wetting their surface. An attempt was made to eliminate this variable factor, viz., workability, by designing the mortar mixes in such a way that water contents per unit surface area of the mixes was constant. By adding to the cement paste, with a fixed water-cement ratio of 0.4 which gave, in earlier tests, a reasonably good workability with medium sand of fineness modulus 2.64 only such quanti-

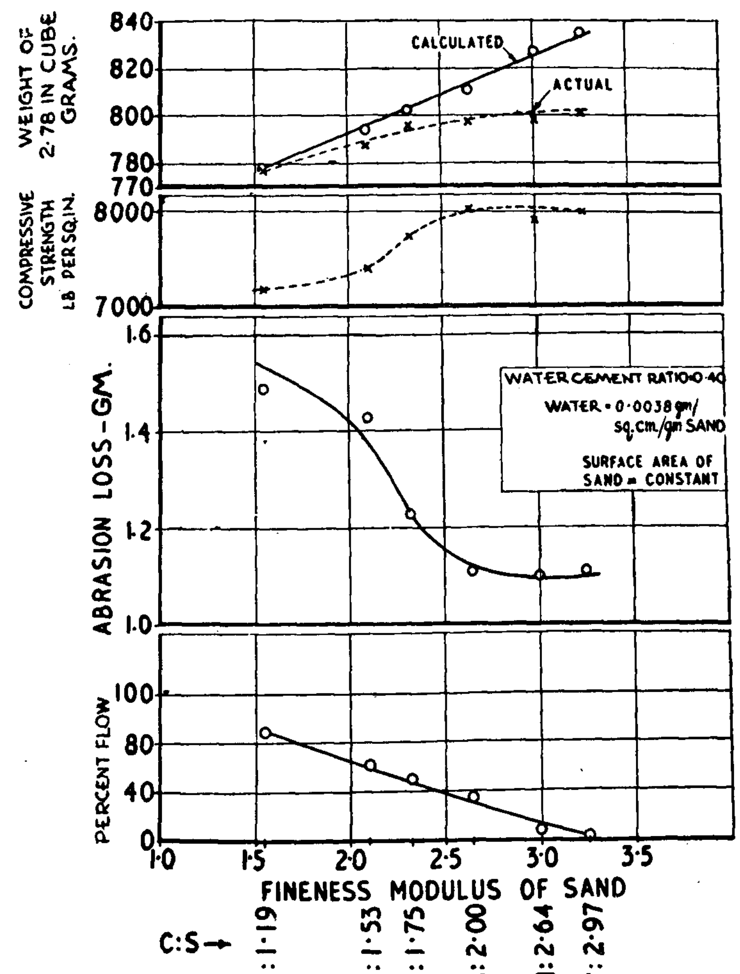

Fig. 5. **Effect of fineness of sand on resistance to abrasion of mortar with fixed (designed) workability and compressive strength,**

ties of sands of different fineness modulii as would give equal surface areas, cement-sand mixes having constant surfaces area to be wetted per unit quantity of water were obtained. The variation in cement-sand ratios from 1 : 1.19 to 1 : 2.97 for fineness modulii of sand varying from 1.54 to 3.25 were thus obtained. Water-cement ratio being fixed theoretically, these mixes should provide reasonably constant workability and also constant strength. Fig. 5 shows the compressive strengths, calculated and actual bulk densities, the percentages of flow and abrasion resistances of these mixes.

It is seen from Fig. 5 that coarser sands with good grading could give better results than finer sands. The substantial economy in cement possible with the use of coarser sands was evident from the values of cement sand ratios with different fineness modulii.

Although an attempt was made to maintain a constant workability by controlling the surface area and the water content of the mixes, it could be seen that the flow had varied from 3 per cent for coarse sand to 85 per cent for fine sand. This could probably be explained by the fact that with coarser particles in the mix the internal friction was appreciably improved.

5.4. Effect of Fineness of Sand with Constant Workability and varying Compressive Strength

In the tests described in paras. 5.2 and 5.3, it was seen that although full compaction was attempted, there existed some difference between the calculated and actual densities of the mortar. This could partly be explained by the fact that the finished cubes might not all have exactly the same volumes. Besides, mixes also contained some amount of entrapped air giving densities lower than the calculated. Workability indirectly influenced the density to a large extent because the stiffer mixes did not lend themselves to 100 per cent compaction.

In order to eliminate the influence of workability, a series of tests using a fixed cement-sand ratio of 1 : 2, sand of fineness modulus 1.54 to 3.25, and water-cement ratios adjusted to give constant workability (a flow of 35 per cent as obtained in the reference mix $C:S=1:2$, $W/C=0.40$ and fineness modulus 2.64) was made. The results of the tests are shown in Fig. 6. It could be seen that although the variation of water-cement ratios for different fineness modulii was not very large (0.38 to 0.45), the compressive strength and the resistance to abrasion has a wide variation. This indicated the important role, the selection of grading and fineness of sand could play in the performance of the pavement,

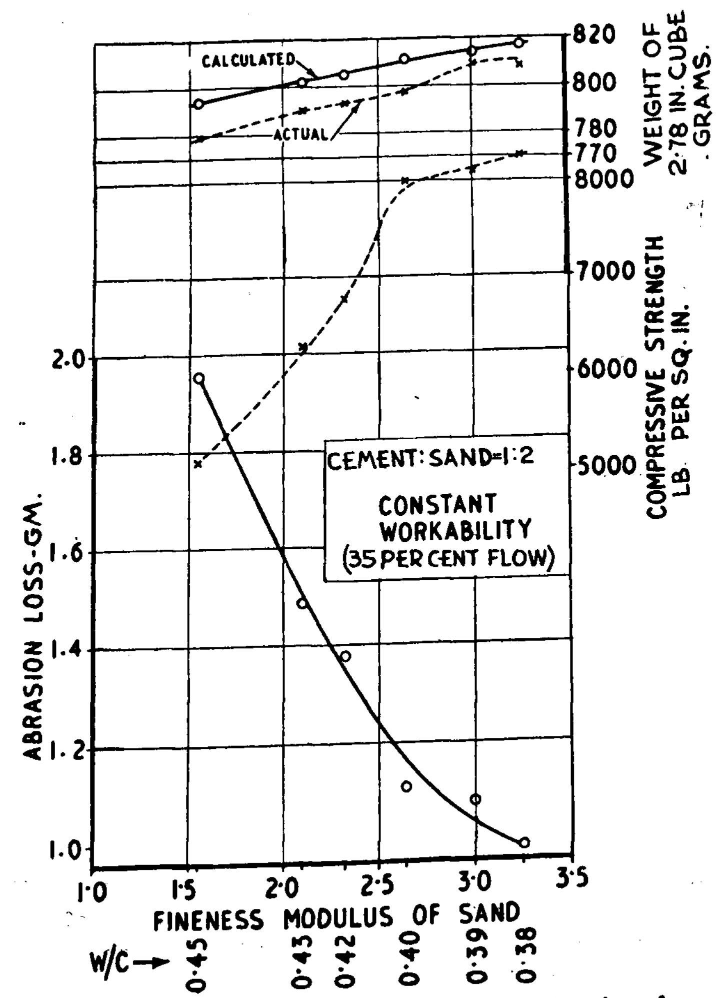

Fig. 6. Effect of fineness of sand on resistance to abrasion of mortars with constant workability and varying compressive strength.

The difference between the actual and calculated densities narrowed down with the use of coarser sands indicating better compactability. The values of abrasion loss plotted against compressive strength Fig. 7, showed a relationship which was distinctly different from the relationship obtained with sand of medium fineness namely 2.64. Superimposing the reference curve of Fig. 2 on Fig. 7, it could be seen that the new line crosses the curve in the reference point of the mix ($C:S=1:2$,

$W/C = 0.40$, $F.M. = 2.64$). Similarly, it was expected that for sands of different fineness modulii, curves relating the compressive strength and abrasion resistance would be more or less parallel to one another and intersect the curve in Fig. 7 at points representing their respective fineness modulii. This would prove

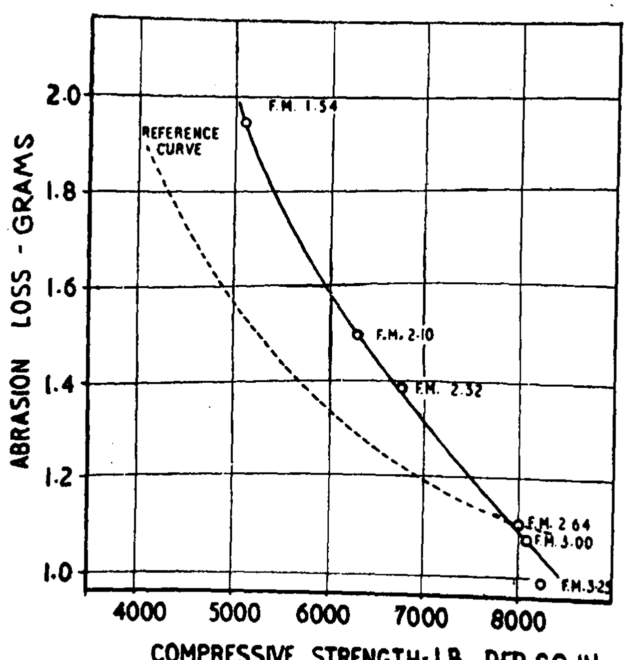

Fig. 7. Comparison of relationship between compressive strength and resistance to abrasion of mortars with sands of varying fineness modulii with that for a mortar with a fineness modulus of 2·64.

that for identical compressive strength, the abrasion loss would be more for finer sands than coarser ones.

6. EFFECT OF DELAYED CURING ON RESISTANCE TO ABRASION

6.1. Effects of Lapse in Curing

Two series of tests were conducted, (a) wet series and (b) dry series, in order to cover probable conditions in the field. In

the wet series, a constant period of dry air curing representing a temporary failure in curing in the field was combined with a continuous or broken period of wet curing totalling 10 days. In order to verify the effect of stopping the wet curing at different ages of the concrete, a 3-day dry curing period was introduced at early, middle and late stages of curing. The results of the tests are shown in Fig. 8.

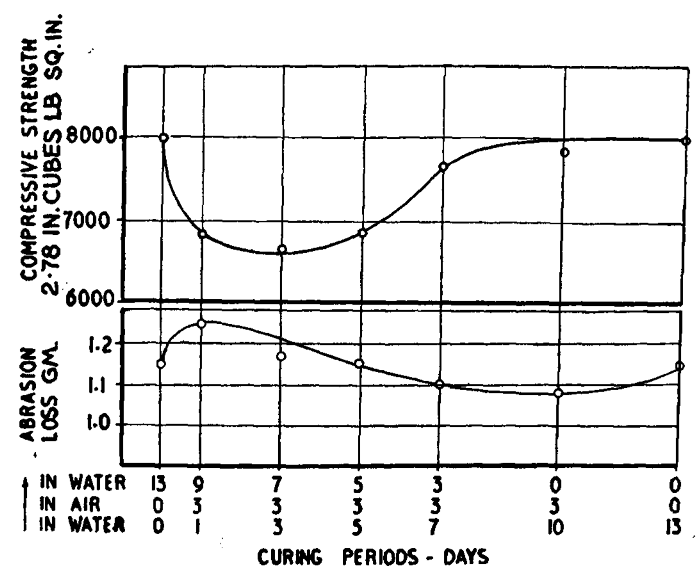

Fig. 8. Wet series—Effect of lapses in curing on resistance to abrasion of mortars.

In the dry series, the effect of lapses in curing was studied by increasing periods of curing in air. Dry air curing periods varied from 0 to 13 days with corresponding decreasing periods of wet curing from 13 to 0 days. 2.78 in. mortar cubes using well-graded sand of fineness modulus 2.64 with a water-cement ratio of 0.40 and cement-sand ratio of 1 : 2 were used. The values obtained in the tests are plotted in Fig. 9.

It could be seen from Fig. 8 that the values of abrasion loss did not change materially on account of temporary interruption of a short duration in wet curing. Interruption in curing in early age of concrete would, however, tend to have a more detrimental effect than when it happened later.

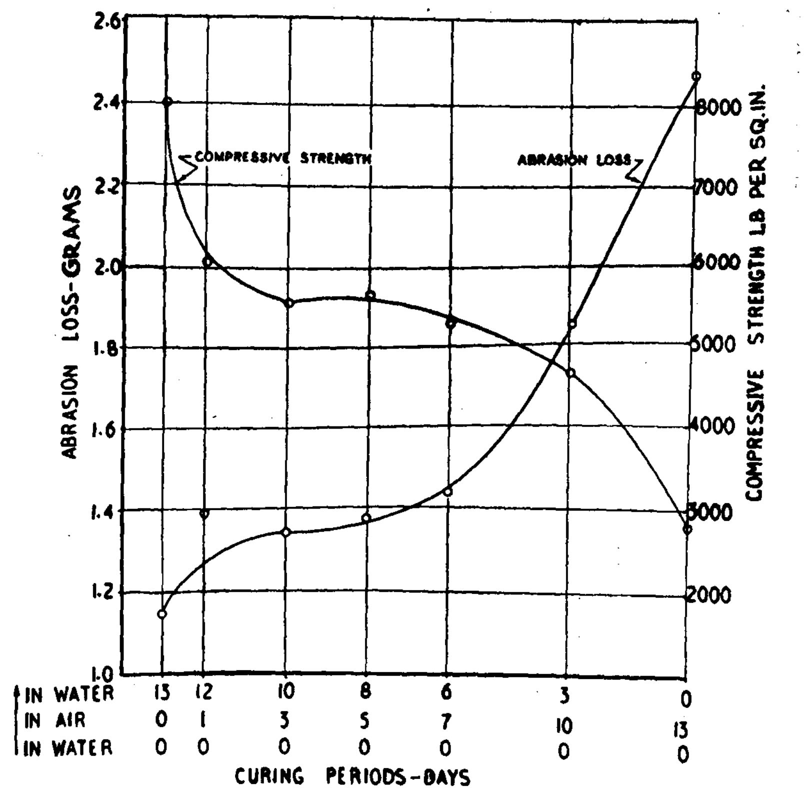

Fig. 9. **Dry series—Effect of lapses in curing on resistance to abrasion of mortar.**

In the case of dry series, Fig. 9, the abrasion resistance was very much reduced, the value of abrasion loss being significantly higher, due to lapses in curing in the early age. Apparently, the damage done by delayed curing even by one day was very pronounced. The abrasion loss increased with increasing periods of delay with corresponding reduction in compressive strengths.

6.2. Effects of Membrane Curing on Resistance to abrasion

In order to find out whether the use of liquid and solid membranes would have any effects on the surface hardening of concrete as compared to wet curing, mortar cubes of 1 part cement and 2 parts sand were covered and sealed with solid membranes or sprayed with liquid membranes by a pneumatic gun at a regulated rate of application per unit area. The membranes were applied two hours after moulding of the specimen. The materials tried were:

(1) Plastic film

(2) Waterproof paper (2 ply) 4 qualities

(3) Wax emulsion (with 25 per cent and 50 per cent dilution)

(4) Asphalt primer.

The trend in the loss due to abrasion, Table 3, in all cases of membrane curing was higher, indicating that wet curing was the best. However, plastic film and wax emulsion membranes and waterproof paper deserved consideration, for reasons of economy, in places where shortage of water and possible lack of supervision and control existed.

7. EFFECT OF ADDITION OF SURKHI ON RESISTANCE TO ABRASION

7.1. There was sufficient evidence to prove that surkhis manufactured by burning certain types of clays at optimum temperatures, when admixed with cement in concrete construction, could yield very tangible benefits such as higher compressive and flexural strengths, greater workability of mix, higher impermeability to water and greater immunity to alkali-aggregate reactivity and sulphate attack. Use of surkhi could also make possible considerable economies by way of saving in cement as it was possible to replace a certain percentage of cement by finely ground surkhi without any loss in strength. It would also reduce the heat of hydration. Therefore, observations on the effect of adding surkhi on the resistance of mortars to abrasion were made.

Fig. 10 shows the abrasion loss with different percentages of replacements of cement by surkhi. The specimens were tested at the age of three months after 14 days of wet and 2½ months of air curing. The ratio of water to cement plus surkhi was 0.40 by weight. Compressive and flexural strength for the various precentages of addition of surkhi were also determined. The flexural strengths were determined on 28-day old specimens and compressive strengths on 3-month old specimens.

The flexural strength increased considerably for 5 per cent replacement. The resistance to abrasion also improved appreciably for 5 per cent, but decreased slightly up to 10 per cent

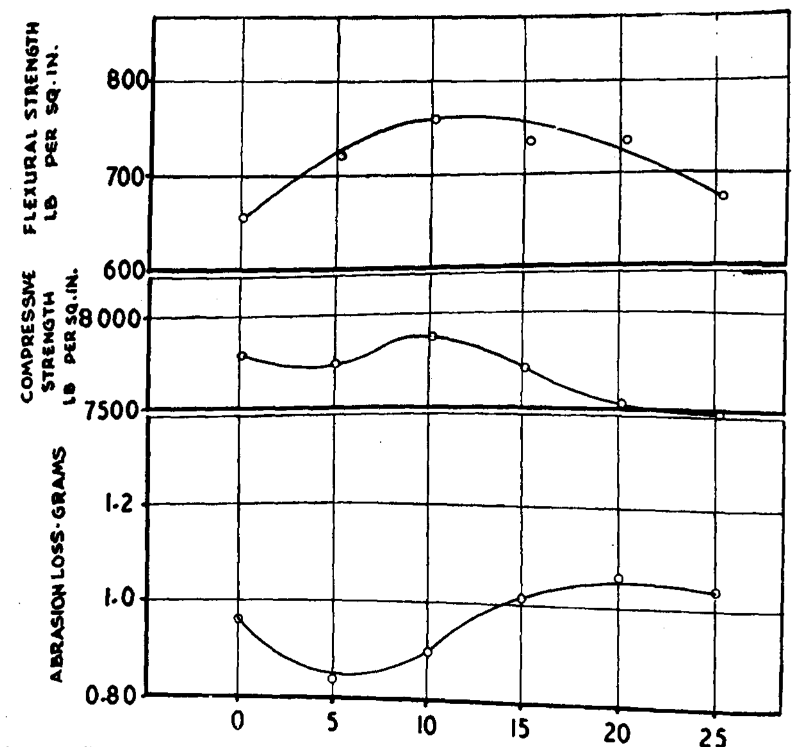

Fig. 10. **Effect of replacing cement by surkhi on resistanace to abrasion of mortars.**

although loss due to abrasion was still less than that of the blank specimen. The resistance to abrasion decreased rapidly for replacements of more than 10 per cent. Different surkhis had varying degrees of reactivities with cement depending upon their minerological composition, temperature of burning and degree of fineness. The results mentioned above obtained with one particular surkhi indicated that there was a possibility of improving the resistance to abrasion of concretes by addition of optimum percentages of surkhis. Also, values of flexural strength and abrasion resistance obtained indicated a possible relationship between each other. But this aspect deserved further study.

8. CONCLUSIONS

8.1. For judging the abrasion qualities of aggregates to be used in concrete pavements subject to heavy bullock

cart traffic, limiting values of Los Angeles abrasion, crushing value and the aggregate impact value specified in foreign specifications would not prove to be satisfactory.

8.2 The Pneumatic test might prove to be a better means of judging the resistance to abrasion directly than through indirect existing tests. It might also provide a more reliable index of field performance.

8.3. A compressive strength of 4,000 lb per sq. in. on 6-in. works cubes was warranted for concrete surfaces subjected to bullock cart traffic. Where there was heavy traffic of bullock cart, compressive strengths higher than 4,000 lb per sq. in. were advisable provided the aggregates to be used were sufficiently strong.

8.4. For identical compressive strength, density of mortar and cement content, use of the coarsest sand, i. e. sand having the highest fineness modulus, showed the highest resistance to abrasion. Where the natural sand was fine, it might be worthwhile to mix coarse crushed rock screenings to increase its fineness modulus.

8.5. With constant strength and workability (the two basic factors in design and construction), use of coarser sands not only provided better resistance to abrasion but also a substantial saving in cement.

8.6. For a fixed workability and cement content, the compressive strength and resistance to abrasion improved very rapidly with increase in fineness modulus of sand. For any change in the fineness modulus of sand, the rate of changes in compressive strength and abrasive resistance was significant even for very small changes in the water-cement ratio which were necessary to secure a fixed workability.

8.7. A temporary interruption of short duration in curing would not materially affect the resistance to abrasion of the concrete pavement.

A lapse in curing of howsoever small a duration during the earlier stages of curing should be definitely avoided.

8.8. The efficiencies of certain membrane curing compounds were comparable to that of wet curing. However, those roads likely to carry heavy traffic of bullock cart should preferably be wet cured.

8.9. The resistance to abrasion of cement mortars improved when cement was replaced in optimum percentages, by surkhi.

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TABLE 1

Abrasion test values of some road stones obtained by Different Standard Tests

Sample No.	Description of aggregate.	Pneumatic Test Loss gm.	Dorry Test Loss gm. per sq. in.	Deval attrition percentage of loss: Dry Test	Deval attrition percentage of loss: Wet Test	Los Angeles Abrasion percentage of loss.	B. S. Crushing Value	B. S. Impact Value.	Specific gravity (S.S.D.)	Absorption (24 hrs.) Percentage
1.	Jalore granite (Rajasthan)	0.50	0.473	2.2	2.1	13.1	16.4	15.1	2.62	0.52
2.	Bengal gneiss (Chaibasa)	0.50	0.465	2.9	3.6	7.3	11.2	8.3	2.92	0.79
3.	Bundelkhand granitic gneiss (Jhansi)	0.60	0.397	1.9	2.1	12.5	18.1	15.3	2.62	0.84
4.	Rajmahal Trap (West Bengal)	0.63	0.686	1.9	1.9	8.5	11.7	10.3	2.77	1.75
5.	Bairenkoda Quartzite (Cuddapah)	0.68	0.350	2.0	2.8	12.3	15.6	17.8	2.62	0.55
6.	Quartzite (Delhi)	0.72	0.436	3.6	2.3	17.8	23.8	22.9	2.66	0.19
7.	Limestone (Chittral)	0.85	2.305	4.7	6.7	13.2	19.2	15.6	2.70	0.38
8.	Deccan Trap (Nagpur)	0.97	1.084	3.6	6.6	14.0	16.5	12.5	2.97	4.12
9.	Deccan Trap (Bombay)	1.10	1.312	3.0	5.8	10.2	13.5	8.9	2.93	0.94
10.	Kushalgarh Limestone (Banswara)	1.12	3.187	6.3	12.2	25.2	26.9	18.4	2.69	0.36
11.	Muth Quartzite (U.P.)	1.22	1.959	5.4	10.2	27.8	22.1	24.7	2.83	0.50
12.	Darjeeling gneiss (W.B.)	1.23	Too flaky for test.	1.1	11.9	21.2	22.4	22.0	2.76	0.94
13.	Deccan Trap (Poona)	1.24	1.337	3.0	14.7	11.3	13.1	14.4	2.92	1.88
14.	Limestone (Balasinor)	1.28	2.121	7.1	17.5	10.7	23.3	18.0	2.61	1.36
15.	Sandstone (Sagar)	1.42	2.704	5.4	12.8	26.2	20.6	22.9	2.46	3.26

TABLE 2

Limits of abrasion test valves for road stones in different specifications

Type of Construction	Water Bound Macadam						Bituminous Construction						Concrete Construction			
	Base Course			Surface Course			Base Course			Surface Course			Base Course		Surface Course	
Specification	Los Angeles abrasion per cent	Crushing value or Impact per cent	Deval dry attrition per cent	Los Angeles abrasion per cent	Crushing value or Impact per cent	Deval dry attrition per cent	Los Angeles abrasion per cent	Crushing value or Impact per cent	Deval dry attrition per cent	Los Angeles abrasion per cent	Crushing or Impact value per cent	Deval dry attrition per cent	Los Angeles abrasion per cent	Crushing or Impact value per cent	Los Angeles abrasion per cent	Crushing or Impact value per cent
A.A.S.H.O.	50			40			45			40-45			40		40	
Road Research Laboratory, U. K.				35 L 25 M 13 H	30 L 23 M 13 H		35 L 35 H	30 L 30 H		25 L 17 H	23 L 17 H		50 L 40 H	40 L 35 H	40 L 35 H	35 L 30 H
British Standards Institution											25			45		30
Madras State Highway Department				35 L 25 M 13 H	30 L 23 M 13 H,	8 L 5 M 2.7 H				25 L 25 M 17 H	23 L 23 M 17 H	5 L 5 M 4 H			40 L 40 M 35 H	35 L 35 M 30 H
Lucknow P.W.D.	45	35		35	28		40	35		40 35 P	30 28 P		40	30	35	28

L=Light Traffic. M=Medium Traffic. H=Heavy Traffic. P=Pre-mix.

TABLE 3

Effect of membrane curing on resistance to abrasion of cement mortar

Sl. No.	Type of Curing	Abrasion loss-gm.	Compressive strength lb. per sq. in.
1.	Wet.	1.15	8004
2.	Plastic film.	1.28	7190
3.	Water proof paper—38 gm.	1.31	6701
4.	Water proof paper processed with kraft paper 60 gm.	1.31	6230
5.	Water proof paper plain processed with 70 gm. kraft paper.	1.33	6375
6.	Water proof hessian paper lined two plies hessian cloth and paper in between hessian cloth of 45 in. × 11 gm.	1.30	6289
7.	Wax emulsion diluted with 25 per cent water.	1.26	5593
8.	Wax emulsion diluted with 50 per cent water.	1.30	5941
9.	Asphalt primer.	1.38	6520
10.	Asphalt emulsion No. 3.	1.42	5130

Appendix 1

THE PNUEMATIC TEST—DESCRIPTION OF APPARATUS AND TEST PROCEDURE

The Apparatus

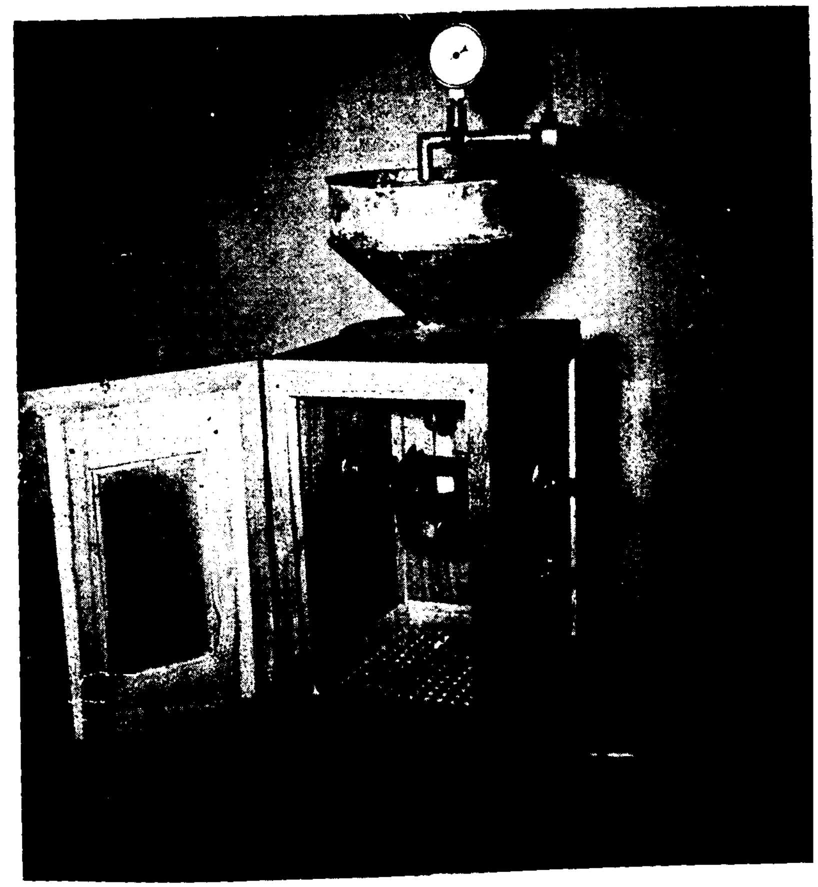
1. The apparatus consists of a wooden chamber, Fig. 11, page 95, and Photo 9, with a tightly closing door having a glass inspection window. A specially designed nozzle, whose axis is vertical, is fitted through the top of the cabinet. A carriage for the test specimen (2.78 in. cubes) is mounted on a rotating trunion passing through the two sides of the cabinet. An arm fixed on one side, on the outside of the cabinet, indicates the angle of inclination of the test face to the direction of the nozzle. Surfaces at any angle of impingement between 30° and 90° can be tested. The axis of rotation of the specimen and that of the nozzle are accurately made to intersect at right angles so that the axial distance from the tip of the nozzle is kept constant with all angles of impingements.

2. The cabinet is provided with sufficient vent area (18 sq. in.) for the escape of air under pressure. The vent windows are fitted with felt sandwitched between wire gauges in order to eliminate dust nuisance. The bottom of the cabinet has perforations, through which sand after abrasion drops into a metallic tray which slides between two guides on the sides with rubber liners to seal the escape of dust.

3. The nozzle shown in Fig. 11 consists of a hollow tube machined out of high carbon steel. For long service life, the tip is made extra-resistant to wear by oil quenching and case hardening*. The tip of the nozzle is replaceable and can be attached to the body of the nozzle merely by a push. Thus, worn-out tips could be replaced easily. The air tube consists of a $\frac{3}{16}$ in. diameter straight tube brazed to a conical steel collar and extends from the open end of the nozzle to a distance of $\frac{1}{2}$ in. from the tip. $\frac{3}{16}$ in. diameter holes, 4 in number, are provided in the side of the nozzle immediately below the collar of the tube. Sand is drawn into the nozzle by suction caused by the high air velocity at the nozzle tip.

4. A conical galvanised iron hopper is provided at the top of the cabinet for holding the abrasive charge. The nozzle is passed through the bottom of hopper and top of the cabinet so

* With 500 repetitions of test the measured wear at the tip of the nozzle was equal to an increase in the diameter of only 0.008 in.



Pheto 9.
Pneumatic abrasion testing apparatus.

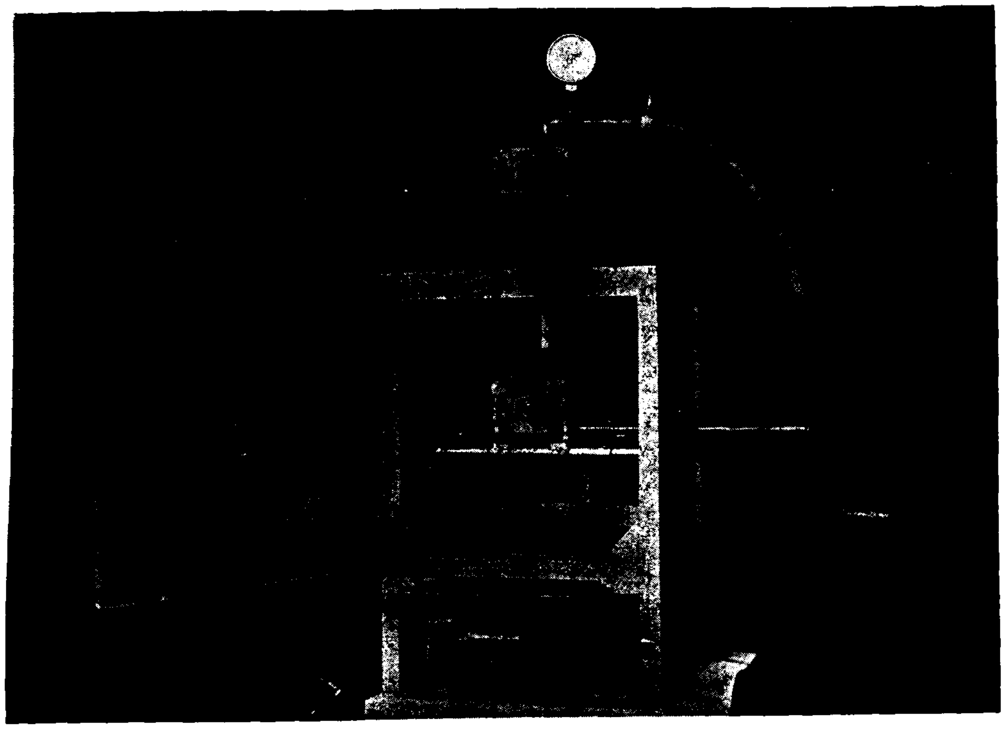


Photo 10. **Modified abrasion testing apparatus.**

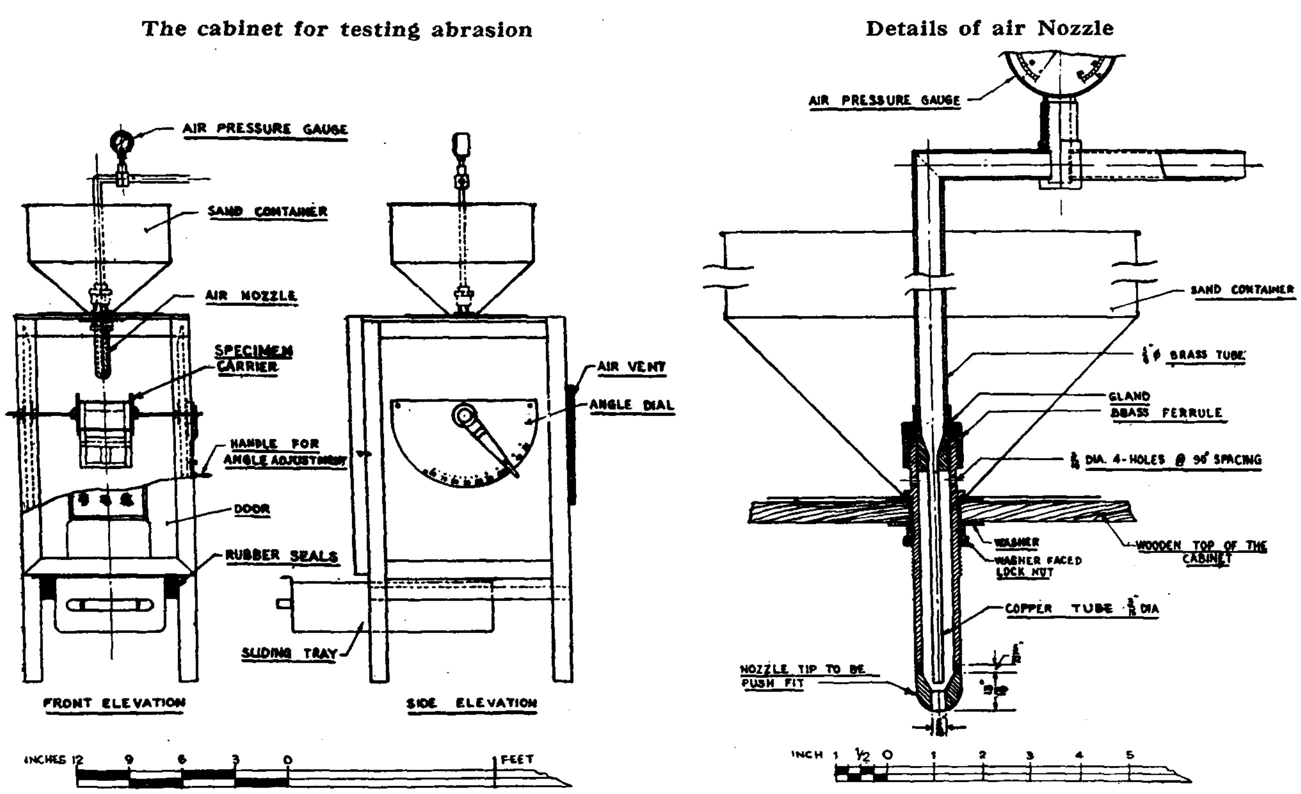


Fig. 11. **Apparatus for testing abrasion.**

that the sand inlet holes go into the lowest position in the hopper. A nut on the threaded portion of the nozzle inside the cabinet fixes the whole assembly. A pressure gauge is fitted to the brass tubing connection to the nozzle to record accurately the air pressure as close to the nozzle as possible. The angle of the cone is such that all the sand runs down the sides on its own as it is being fed into the cabinet. The brass tubing end is connected to a compressor through a balancing air storage bottle by rubber tubings.

The Modified Apparatus

5. The cabinet described above was used in the first phase of the study, namely, standardizing the method for measurement of resistance to abrasion. Different sizes of charge, various air pressures, and angles of impingement were tried. It was not used in the measurement of abrasion resistance of concrete specimens as it was necessary that a larger area of the surface be tested. In order to adapt this apparatus for testing concrete, certain modifications have been incorporated in which a larger size specimen could be tested in a cradle, which would be movable below the nozzle between two fixed points, Photo 10. The angle of the cradle support is so fixed that the face of the concrete specimen is presented to the direction of the nozzle jet, at an angle of 80°. This angle was fixed from the earlier standardization study. The modified apparatus will also lend itself for use in testing the abrasion of concrete specimens extracted from road surfaces in use.

Standard Procedure

6. The following operating conditions were fixed taking into account factors such as easy adaptability of the test, accuracy and repeatability :

1.	Operating pressure	20 lb per sq. in.
2.	The type of charge	Ennore quartz sand, single size No. 18-25.
3.	Amount of charge	2,000 gm.
4.	Axial distance of test face from nozzle tip	2.0 in.
5.	Angle of inclination of the test face	80° from direction of impingement.

Testing of Stone Aggregates

7. In order to facilitate mounting of the test specimen in the carrier of the cabinet, a thin slice of stone is cut. The cube mould is filled with 1 : 1 cement: sand mortar of average consistency and the slice of the stone grafted on the top of the mortar by pressing it down till the test face of the stone lies flush with the top of the mould. The face of the stone slice is left proud of the mortar surfaces by about ¼ in. so that during the test the sand particles do not abrade the mortar.

INFORMATION SECTION

THE TENTH INTERNATIONAL ROAD CONGRESS, ISTANBUL

A delegation consisting of Shri K. P. Mathrani, Consulting Engineer to the Government of India (Road Development), and Shri K. K. Nambiar, President of the Indian Roads Congress and Chief Engineer (Highways), Government of Madras, attended the Tenth International Road Congress held at Istanbul from the 26th September to 8th October 1955. The members of this delegation participated in the various technical discussions of the Congress, specially those which were of particular interest to India, viz., soil stabilization, low cost roads, geometric features of roads, and design of bridges. They served on some of the Committees to draft the general conclusions. The conclusions arrived at the Congress, along with five questions discussed, are given below :

The following are the conclusions reached during the various sessions :

CONSTRUCTION AND MAINTENANCE

First Question : Surfacing of Roads and Runways

(a) Concrete Roads and Runways

1. The materials employed for bases, their stability, drainage, regularity, etc., come within the scope of soil mechanics and are not dealt with in detail here.

Two tendencies are noticeable in connection with the thickness of bases beneath concrete slabs. On the one hand, thick bases up to $1\frac{1}{2}$ times the thickness of the slab are in use under concrete roads, on the assumption that these reduce the stresses in the slabs caused by movements in the subsoil. On the other, it is widely felt that thicknesss of base has no effect on the performance of concrete road slabs, and bases are made thick enough only to carry construction traffic, prevent mud-pumping, and prevent frost damage to the subsoil. The reasons for the divergence require further investigation.

2. The tendency is to use dry mixes but to take into account the necessity for workability in relation to the machines at present available. Attention has been drawn to the need to define the water-cement ratio more accurately in relation to the absorption characteristics, slope and grading of the aggregate proportions of the mix, and the site conditions.

3. The width of slabs is normally between 3 and 4.5 metres, although sometimes it has been increased to 7.5 metres. The spacing for contraction joints in unreinforced slabs is about 5.6 metres and for expansion joints is normally 14.85 metres. Techniques to eliminate the joints or to reduce their numbers are coming into use.

Full scale experimental roads have been laid in many countries to examine the relation between amount of reinforcement and joint spacing, the performance of very thin runway slabs reinforced at the mid-depth, and the use of very thin prestressed slabs on a flexible base. Sections up to 400 metres long have been laid without contraction joints, while in some cases expansion joints are omitted and contraction joints only are used.

4. In runway work, double-slab construction has been successfully used for thicker runways. In road work two-course construction has been adopted in a few instances but in the majority of cases single course work is used.

5. Construction methods for joints have been improved. Adhesion to the concrete and elasticity are important properties for both roads and runways. Runway joints are provided with special filling compounds to resist the fuel spillage, blast, and heat from jet aircaft.

6. Joints are prepared both in the plastic and in the hardened stage of concrete. Bars and joints vibrators serve to prepare joints in the plastic stage. In the hardened stage, excellent results have been obtained by cutting the joints with circular saws.

7. It is generally considered that dowel bars are necessary and some countries have increased the number and diameters of dowels. Where dowels are not used, the bases may be thickened or sleeper beams used beneath the joint.

8. Reinforcement of the mesh type is widely used, the amounts varying between about 1.5 and 8 kg per sq. metre. It is recognized in the majority of cases that reinforcement is useful in preventing the widening of cracks. Where reinforcement is not used it is usual to increase the numer of contraction joints or to increase the thickness of the slab or of the base.

9. The causes of surface scaling are attributed to the effect of frost action and to the use of de-icing salts. Air-entraining agents are extensively used to prevent scaling in most countries where repeated freezing is common, but in Sweden air-entrainment has not eliminated surface scaling although it has reduced

it. The amount of entrained air varies between 3 and 6 per cent. These agents and certain plasticizers are used to increase workability of concrete. The reduction in strength which air-entrainment may cause ought to be taken into account.

10. Prestressing is emerging from the exprimental stage and new prestressing devices have been recently developed. Several countries have reported that it is no more expensive than ordinary pavement construction. It has been applied in the construction of thin jointless runways, and the use of very thin prestressed concrete on top of flexible runways is being tried. Recently experiments have been made with concrete prestressed by means of expansive cement.

11. There have been improvements in the techniques of controlling concrete quality and in the operation of batching plants.

12. Many countries are now using membrane curing. One or two countries still prefer the damp sand method.

13. It is important that detailed information on the design details (thickness of slab and base, weight and position of reinforcement, use of load transfer devices in joints, spacing of contraction and expansion joints) on different kinds of subsoil should be collected from as many countries as possible, and the consideration of such information might well form a subject for discussion at a future Congress.

14. Investigation of the causes and methods of prevention of scaling is an important matter, and it is suggested that a Question on this subject be considered for discussion at some future Congress.

(b) **Surfacings made with Plastic Binders**

15. It is necessary to provide a base which is adequate to carry the heaviest loads under the worst moisture conditions likely to occur. Infiltration of water and ponding on the surface must be prevented by waterproofing, surface drainage, etc.

In many countries lean concrete, stabilized soil, and thick or multiple course coated macadam are used in foundations for flexible surfacing.

16. The choice of materials for the construction of surfacings and foundations shows a tendency to use mainly materials of continuous grading which form dense courses that often prove relatively cheap. Materials having too high a proportion of clay are to be avoided because they are too greatly affected by the

action of water. Reference has been made to the risk of laying open textured surfacings on impervious foundations because of the retention of water in the surfacing.

17. All types of bituminous binders have been widely used. Tar has been the subject of studies concerning the improvement of its resistance to weathering. Tars of higher viscocity have been produced. Tar has been used for grouting surfacings and for coated materials. Satisfactory results have been obtained in some countries with dense tar surfacings made with special tars. On airfields tar has given good results under the action of fuel spillage and jet blast from aircraft.

18. Penetration grade bitumen is used mainly for hot mixes. General use is made of cut-back bitumen, especially for surface dressing, but if conditions do not permit rapid evaporation of the solvent, failure may occur. The use of activators can extend the range of condition under which cut-back bitumens can be successfully used.

Bitumen emulsions are largely used for surface-dressings, repair work, grouting, coating of macadam-type materials, and soil stabilization. A wide range of emulsions of varying stability, rate of breaking, etc., is now available for every kind of application. Acidic emulsions are found to adhere well to siliceous aggregates.

The additions to bitumen of a small amount of rubber may modify its physical properties by conferring a longer-lasting plasticity which reduces brittleness and temperature susceptibility. These effects can be destroyed by over-heating, and they depend very much on the nature of the rubber. The practical application of these results is still in the experimental stage.

19. Coated materials made with bituminous binders are increasingly used. Mobile plants of large capacity for mixing hot materials give accurate control of mix proportions, and can be used for the manufacture of dense graded materials, which are now used in many countries. There are several processes for coating wet aggregates, generally with the use of additive or activators in the bitumen or of lime with the aggregates.

20. In surface-dressing efforts have been made to improve the quality and accuracy of the operations by improving the mechanical equipment. The process of surface sealing consisting of the spreading of fine sand and filler mixed with binder and an appropriate quantity of water continues to be used, especially on airfields (Schlaemme or slurry process). This process appears to show possibilities of development.

21. The effectiveness of activators in improving adhesion in wet weather is recognised, especially with cut-back bitumens. However, an excessive proportion of activator must be avoided, and care must be taken that the effects are not impaired during heating or by prolonged heating.

22. Rock asphalt, with or without the addition of bitumen gives surfacings of good quality.

23. For a discussion of stripping tests, reference should be made to the Report of the Technical Committee on the Testing of Road Materials. It is not yet possible to propose a standard method, as this very complicated question is still being investigated by the Committee.

24. The Drafting Committee in prepraing this part of the conclusions has encountered many difficulties of exact translation due to differences in terminology between the different languages, and even within the same language. It is suggested that the 6-language Dictionary be expanded by including many more definitions, and that consideration be given to standardizing terminology.

(c) Questions Common to various Surfacings

25. To obtain and maintain a sufficiently high co-efficient of friction is an absolute necessity in road construction. The Committee on Slipperiness has already published a full report on this subject.

26. Ice-formation on roads can be combated by :

(a) Applying abrasives or chemicals by machine ;

(b) Improving the equipment used for the removal of compacted snow.

A method of making the surface water-repellent is mentioned in one report as being the subject of laboratory tests.

27. A general tendency is noted towards increased use of mechanical equipment for the construction and maintenance of roads. This cannot be analysed in detail, but two main motives can be indicated. One is that of reducing the cost, and the other that of improving the quality of the work done, especially in improving the uniformity of the surface.

Second Question : Soils

The comments on major problems concerning bearing capacities and soil stabilization, contained in 15 reports, prompts the drafting committee to summarize the whole subject in eight items of major importance.

1. **Preliminary Surveys**. As the earth formation is one of the basic factors that affect the bearing capacity of soils, a survey and study of the geological, geotechnical, and pedological formation of land where the desired highway is to be located is essential.

Due weight should be given to the results of such surveys and of local experience in deciding upon the alignment for a new road.

2. **Drainage**. This provides a means of improving the bearing capacity of soils by controlling the underground water table levels and minimising the possible percolation of water through the pavement surface and verges.

Subsoil drainage can be objectionable, particularly in some cases of highly compressible soils where it may result in uneven settlements.

Vertical sand drains have been successfully applied to accelerate consolidation of some soils under embankments.

3. **Frost Effects**. Frost damage results primarily from bad drainage, but it also depends upon the type of soil and its state of compaction.

4. **Compaction**. Good compaction is essential for improving bearing capacity and stability of soils and base materials. The latest types of vibratory compactors are proving very effective on non-cohesive materials. Note should be taken of the new method proposed by Great Britain for controlling the state of compaction in terms of the air content.

5. **Bearing Capacity**. Bearing capacity tests and methods of computation are still based on empirical methods. Different methods of design involving the use of penetration tests, plate bearing tests, shear tests, tests for the plasticity properties and other tests still need further studies and investigations for their correlation.

Of the various methods of design of the road structures, the C.B.R. method is the most widely used and is generally found to be the most satisfactory. Some countries have introduced modifications to the original method conforming to their particular experience,

Some give credit to methods based on soil classification and identification adopted to local conditions. It is also necessary to take into account the frost susceptibility of the soil in certain countries.

6. **Selected Materials**. While the grading of materials and the plasticity properties of their fines have an important bearing on their suitability for bases or sub-bases, it must be recognised that their suitability cannot always be assessed on this basis (laterites, volcanic tuffs, etc.).

7. **Rigid and Flexible Pavement Design**. There is a marked difference, as far as the bearing capacity is concerned, between rigid and flexible pavement design methods. The difference arises from the types of pressure distribution under the flexible pavement and the rigid pavement. The pressure distributions for the two cases are of different character.

Natural movements of the subgrade soil due to swelling and shrinkage, climatic conditions, weight of the slab and the base courses, may play a greater part in a rigid pavement design than the bearing capacity of the subgrade soil (which is of great importance for the flexible type of pavement).

8. **Soil-cement, Soil-bitumen, Soil-hydrated Lime Stabilization.** Soil-cement and soil-bitumen stabilization, as far as the technique is concerned, have been widely adopted in some countries and quite satisfactory results have been obtained. If the required construction equipment is available, and the cost of cement or bitumen is within economical limits, they can be applied with confidence to many soils.

Stabilization with hydrated lime has been successfully applied in some countries, but there is a need for more experience on this subject.

9. **Field Observations**. Carefully conducted field trials are a valuable means of studying the problems considered on this subject.

Third Question: Low-Cost Roads

(a) Lightly Trafficked Roads in Rural Areas

1. Analysis of questionnaires indicates a growing interest in the use of local materials. More use is being made of cheap local materials for bases and surfacings.

2. The main function of lightly trafficked roads is to provide access to main arteries rather than to allow high speed travel.

3. Soil stabilization is successfully employed in the construction of low-cost roads.

4. The suitability of materials for this purpose should be properly determined.

5. In some countries soil tests have not yet been widely adopted to indicate the thicknesses of bases and surfacings required for this type of road. The development of a simplified testing procedure for use both in the design of new roads and to indicate the amount of strengthening required in existing roads would be most valuable.

6. It is essential that the road-bed should be well drained, because no road made of fine materials can withstand prolonged soaking.

7. The thickness of the courses is governed by local carrying capacity of the subgrade, climate, ground-water level, possibility of drainage, and intensity and volume of traffic.

8. More attention is now being given to the control of moisture, both by the development of methods to waterproof the formation during the construction period and by controlling the water table to ensure stable conditions in the subgrade of the new road.

9. The limit for economic maintenance of low type surface, such as sand, clay, gravel and other types of local materials, is 50-200 vehicles per day, depending on local conditions.

10. Dust prevention is an important problem in dry climatic conditions because of the blowing off of the materials. The dust is a nuisance to the road users, causes hazard to traffic and damages the agricultural products, etc.

11. Under certain climatic and local conditions when it is economically feasible it is recommendable to add hygroscopic matter, e.g., calcium chloride, magnesium chloride, sulphite lye, waste salt, etc. to the soil mixture in surfacings, to provide a dustless surface.

12. A high formation line, above the general level of the terrain is desirable in regions of sand and snow drift.

13. Continuous manintenance protects initial investment by forestalling progressive deterioration and insures retention of safety features.

14. Maintenance, whether mechanised or not, must be varied to suit climate, location, materials and traffic.

15. Road maintenance can be considered as of two categories: routine, and seasonal maintenance. In order to ensure maximum economy, annual maintenance should be organised in relation to the natural and seasonal characteristics.

16. The quantity of materials to be added each occasion for maintaining the road surface depends on the climate, nature of material and the traffic intensity.

17. Efficient maintenance combined with some improvements can considerably improve the road in a few years.

(b) Roads in Under-developed Areas

18. Economy of construction implies that the road should be designed to the maximum load it has to carry—when the maximum load has to be increased the thickness of the base has to be increased accordingly. Stage construction must, therefore, be applied. Maximum axle load must be determined according to the strength of the road.

19. Corrugation can be effaced by repeated passes with light graders or drags equipped with blades or old tyres. Also by scarifying, levelling and compacting with pneumatic-tyred rollers.

20. The most important development in the construction and maintenance of earth roads has been the increased use and improved efficiency of mechanised equipment.

21. Two advantages can be obtained by mechanisation in road construction and maintenance: (a) the speed and low cost of work, (b) the quantity and the improved quality of work.

22. Mechanisation of road construction and maintenance become most economical if the plant is fully utilised.

23. Stage construction must be considered in the design of low-cost roads.

24. The following should be considered in the design of low-cost roads: (a) traffic density; (b) topographical conditions of the terrain; (c) engineering properties of available materials; (d) drainge conditions; (e) moisture control; and (f) possibility of future development.

25. The essential functions of roads in under-developed areas is to create at a low cost an economic network.

The roads should have high geometrical standards to provide low cost transportation, safety and fast communication.

26. One of the primary objectives is to make the road permanent and eliminate temporary engineering structures.

27. Low cost can be achieved in base courses and in surfacings by using local materials. In special cases bituminous surfacings might be used where it is economically feasible.

28. The type and volume of traffic must be taken in consideration in determining the cross-section and type of surface.

29. No feature of a road has a greater influence on the safety and comfort of driving than the width and condition of the surface.

30. Sight distance was found to be one of the most important factors in the safe and efficient operation of a road.

31. For single-lane roads effective widths of 3 to 3.5 metre surfacings are considered adequate with wide enough shoulders to provide passing of cars coming from the opposite direction. For two-lane roads, effective surface widths within the range of 6 to 7 metres are considered.

32. Shoulders should be wide enough to prevent hazard to traffic. If possible 1.5 to 2.5 metre shoulders are recommended.

33. When soil conditions permit, "V" shape ditches are recommended.

34. Cut-and-fill slopes depend on the characteristics of soil, drainge and also on the height of fill and on the depth of the cuttings.

35. The roads are normally cambered to drain from the centre line except where super-elevated.

36. The choice of minimum radius is influenced to a considerable degree by the character of the terrain.

37. A small radius of curvature is not necessarily objectionable, unless it restricts sight distance or creates hazard to traffic.

38. Consistency is the most important element in curvature.

39. The physical characteristics of the site, characteristics of traffic, safety conditions and economic considerations generally determine the actual maximum grade. Projects should aim at consistency in maximum of grade.

40. The minimum grade is determined by drainage conditions.

41. The exact location of a road is a problem of engineering economics that is solved by an analysis of location, inspections and surveys.

42. Surveys should be entrusted to qualified and experienced personnel.

43. Aerial photography and aerial survey techniques are of considerable assistance in studying alternative locations, and soil types.

44. Headroom below bridges should be determined with due consideration to trees, boulders, timber, stumps, ice, etc., which drift or move in the stream during flood stages. At river crossings requirements of navigation should be considered.

45. A minimum vertical clearance of four metres is recommended for underpasses and tunnels.

46. Bridges must be designed to allow carrying capacity sufficient to permit of safe haulage of equipment and supplies.

47. Since the bridge is not a progressive creation like the road, it is advantageous to construct all bridges to a final standard, applying stage construction to roadwork only, except in special cases where temporary superstructures may be considered. (Applies only to under-developed areas).

48. It is recommended that the width of bridges should at least match the width of carriageway, when the economic conditions permit, minimum one-metre sidewalks should be considered in the design of bridges.

49. The cost of maintaining roads is directly related to the adequacy of the means provided for drainage.

50. Good drainage design depends on anticipating where surface run-off or ground water will occur, and how often, and making provision for removal of excess water as rapidly as is necessary to avoid undue interference with operation of vehicles or excessive cost for maintenance.

51. There is a lack of an adequate method for calculating free openings of bridges and structures.

The effect of any forest clearance on the catchment area may greatly increase the run-off intensity and consideration should be given to this possibility.

52. Highway engineers are frequently obliged to make their calculations for bridges and drainage structures on the basis of observations made on the performance of drainage structure on existing roads in the same district or on local information.

53. Tolerable discharge rather than maximum discharge should be used when determining required waterways of drainage structures. Foundations of structures, when they do not rest on rock or hard unerodable material should, however, be designed to withstand the maximum scour that may be expected under the worst possible conditions.

54. Frequency analysis is favoured in the determination of waterways, giving due consideration to the type and importance of the road.

55. Statistical methods are found useful in evaluating short-term run-off and rainfall data and comparison of hydrological data of similar regions or countries. Exchange of information between countries on this subject is considered a further help in reducing costs of bridges and culverts.

56. Small culverts and bridges should be standardized to the maximum possible degree.

57. Maintenance, whether manual or mechanical, is an important and complex problem which involves a high cost. Studies of new methods for maintaining low-cost roads should be carried out.

Definition of Low-cost Roads in Under-developed Area

"A low-cost road is one which, having regard to considerations of climate and traffic, has been located and built to geometrical standards commensurate with future requirements, but it has been constructed with bases and surfacings to meet the present traffic requirements. It is, however, one which should be so designed, constructed, and maintained that it allows for stage construction when traffic requires it, and improvement in economic conditions permits."

ROADS IN RELATION TO TRAFFIC ADMINISTRATION AND ALLOCATION OF FUNDS

Fourth Question: The Road in Relation to Traffic

(a) Traffic Circulation

1. Plans for highway improvements should be based, whenever possible, on exact knowledge of requirements to be met, and

from a general standpoint, on statistics relating to traffic and its composition, and the determination of laws governing traffic movement.

2. Information about the nature, volume and weight of traffic using the roads is necessary for a sound and economical system of highway administration and maintenance. In particular it is desirable to make traffic surveys before and after changes are made in the road system or in traffic control.

3. Manual traffic counts alone have been inadequate for the thorough studies of traffic, and it is recommended that these are supplemented by introducing permanent and systematic counts by means of automatic mechanical counters.

3a. In several instances, 5-hour counts have proved to be adequate for a fair estimate of average daily traffic.

4. Traffic surveys and studies like weighing counts and origin destination studies have proved well worth while in the past, and should be extended as much as possible in the future.

5. Hourly traffic data for various classes of roads should be investigated, if not already available.

6. Estimates of future traffic should take into account local economic developments, changes in vehicle ownership, and the travelled vehicle-kilometres. More accurate information should be collected on the total vehicle-kilometres travelled.

7. More research is desirable on the amounts of traffic generated when new traffic facilities are provided. Until further knowledge is available, however, it is useful to assume that traffic generated increases with the product of the populations of the places connected and that it decreases as their distance apart increases.

(b) Road Characteristics in Relation to Traffic

8. Technical aspects of road design must be reviewed in the light of their effects on the safety, economy and convenience of traffic.

9. Continuity and uniformity should be the predominant features of road characteristics governing alignment, design, planting, pattern of the road network, and riding.

10. Geometric standards relating to the layout and the profile of the road, such as horizontal and vertical curves, gradients, superelevations, etc., are particularly dependent on design

speeds, which vary according to the classes of roads and types of terrain, and cannot be standardised uniformly for all countries.

11. A gently curved layout with horizontal and vertical curves well blended should be given to the road, in preference to very long straight sections.

12. More emphasis should be laid upon the study of road design in three dimensions, which is a new theory that can be supported by means of models and use of multi-coloured images.

13. In the design of highways, special consideration should be given to the dynamics of riding, and necessary investigations dealing with this subject should be undertaken.

14. The design of the cross-sectional profile of the carriageway, in particular the number of traffic lanes and their widths, depends mainly on the peak-hour traffic that has to be handled and its composition, and each country's needs should be determined, taking into account local traffic conditions.

15. Although various capacity criteria are used at present in different countries, further road capacity studies are needed to determine thoroughly the actual situation and establish the number and width of traffic lanes according to local traffic requirements.

16. On roads having a high design speed or carrying a high proportion of commercial vehicles, it is desirable to have lanes with a minimum width of 3.5 metres.

17. Under existing circumstances, thorough investigations should be made from the aspect of expenditure, safety and convenience of traffic, before condemning 3-lane highways.

18. In the distribution of vehicles across the carriageway, the separation of traffic lanes by means of directional islands, carriageway markings and reflecting studs, proved to have resulted in a marked improvement, but further observations should be made on this subject.

19. In order to increase the capacity of the road, measures should be taken, wherever practicable, to segregate from the main traffic pedestrains, animal-drawn carts, pedal-cycles, cycles with auxiliary motors, motor-cycles, and certain other categories of road users.

20 Duel carriageways, divided by a median strip, are recommended when traffic reaches a high level.

21. An aim of junction and intersection design should be to minimise the relative velocity between merging traffic streams, and to increase up to 90 degrees the angle at which different traffic streams meet. A simple compact layout and adequate visibility should be the essentials, for both road safety and flow of traffic.

22. In the interest of traffic movement it is recommended that the highway engineer restricts his range of intersection layout systems to a few standard arrangements, so that road users can grasp easily what is required.

23. With moderate volumes of traffic, roundabouts, variously-shaped islands and refuges can often be advantageously used at intersections and junctions. Where an even flow of large volumes of traffic at high speed is desirable, intersections should be eliminated by grade separations. In certain cases, the adoption of directional islands combined with traffic signals may provide a good solution. The problem is complicated and each case must be considered on its merits.

24. There is no uniformity between the various types and sizes of roundabouts, islands, refuges, etc., being used at intersections in different countries at the present. It is recommended that further studies should be undertaken in order to determine the actual capacity and traffic requirements of such designs.

25. Level intersections should be eliminated on motorways (autoroutes) where traffic is reserved to motor vehicles, and this system should be applied wherever possible.

26. The adoption of a generally recognised measure indicating the degree of traffic congestion is desirable.

(c) Road Equipment (Outside Built-up Areas)

27. An international standardisation on a world-wide basis of road signs and signals is desirable.

28. Lane markings, traffic beaconing and reflecting studs have proved valuable and should be adopted as standard methods for dividing the carirageway into traffic lanes and indicating the course to be taken at junctions and curves.

29. Where drivers may be in doubt as to the limits of the carriageway, special provisions should be taken to mark the edges of the carriageway which enhances the capacity of the road.

30. In order to make signs legible in the dark, they should be either fitted with reflecting materials or illuminated externally.

31. Emphasis should be laid on the value of properly-designed highway shoulders for parking purposes, and lay-byes should be provided where there are no shoulders, on the condition of effective signing.

32. Special consideration should be given to the question of parking at scenic points.

33. The setting up along important routes of a system of telephone call-boxes at the disposal of road users, bearing a standard sign and equipped with necessary information, is essential.

34. First-aid posts should also be established at regular intervals along highways, with special signs to draw attention, and containing first-aid kits for use in cases of accident.

35. Travelling road patrols should be set up to render services to road users.

36. Radio-telephone installations should be provided for communication, in regions where winter conditions are severe.

37. The establishment of a road weather service, informing highway users about the conditions of the road network in winter time, and broadcasting by radio, is recommended.

38. Service stations are an integral part of the traffic system and should be included in the planning of highways, and be designed to meet future needs.

39. Service stations should be designed not to interfere in any way with road safety. The frequency, siting, layout, appearance and facilities of these stations should be determined according to each country's local conditions and needs.

40. The exertion of a certain amount of Government control over the somewhat haphazard methods of private enterprise in order to obtain a better distribution, siting and layout of service stations is desirable.

41. Hotels, inns, motels, restaurants, roadside cafes, and the like, should be established along or near highways for the convenience of road users. However, this recommendation does not apply to short routes designed to avoid built-up areas. A strict control should be exercised by competent authorities over the siting and layout of such establishments.

(d) **Accident, Road Safety**

42. Accident frequencies have been reduced by action based on the study of accident statistics, and it is recommended that much more should be done on these lines in the future. In particular, research should be carried further to determine accident rates of various types of severity corresponding to different types of roads and junctions, and the effects on accidents of road improvements and human factors.

43. Accident spot maps and stop plans have proved very valuable, particularly in redesigning of dangerous road intersections, and should be kept up as complete as possible.

44. The comparison of accident experience in different countries has yielded good results, and it is recommended that accident data should be compiled on a uniform manner and based on standard definitions.

44-a. The work now being done by the Economic Commision for Europe is valuable and should be used as much as possible.

45. Highway and traffic engineers, the police, research institutions and all other interested authorities and bodies, including education authorities and associations of road users, should work in close co-operation with a view of improving road safety.

46. Where justified by local physical and traffic conditions, it is desirable to adopt speed limits within built-up areas, and strict control of speed limit violations should be exercised by competent authorities.

47. Where accident rates per kilometre travelled are greater in winter months, winter service should be made more effective to reduce glazed ice and slippery snow coverings on the surface of the roads.

48. To reduce accidents, sections with anticipated high amount of truck traffic should be provided with at least one slow-moving vehicle lane, wherever practicable.

49. The road gradient should be adjusted to the engine capacities of the heavy traffic, and it is desirable that on main roads the maximum gradient does not exceed 4·5 per cent, and that the maximum gradient and minimum radius do not coincide to avoid heavy skewed inclines.

50. In order to provide sufficient space for parked vehicles and reduce the risk of accidents, shoulders on main highways

with large amounts of traffic should be constructed with a minimum width of 2·5 metres, wherever applicable.

51. Raised kerbs with vertical front faces are dangerous on highways, and should be used only where there is a considerable volume of pedestrian traffic.

Fifth Question : Allocation of Funds and Economic Justification of Road Work

(a) Methods of Financing Works

1. A classification according to function of all the roads in a country should be the first step towards establishing a method of financing.

2. The development of transportation and the reduction of its cost leads to a higher level of general economic activity and to an enrichment of the country as a whole. In under-developed countries the greater share of the burden of road maintenance and construction is therefore borne by the general budget.

3. As the volume of traffic increases, it becomes possible to shift this burden gradually to the road-users. But road-user tax revenue should, as a first priority, be spent on roads to the full extent required by the needs of highway traffic.

4. In view of the time required by road works, these needs must be evaluated not only on the basis of existing traffic, but also taking into account expected traffic in the foreseeable future. Road appropriations should therefore constitute a specified proportion of total revenue derived from highway traffic so as to permit the drawing up of long-range programmes and to ensure their regular execution. Attention is drawn to the fact that these circumstances are precisely those which enable road work to be executed at the lowest cost.

5. The use of loans must be considered as justified when it comes to the rapid execution of road works of exceptional importance in relation to ordinary and permanent funds available to highway departments. This applies particularly where important backlogs have arisen in the development of highway networks, or when a large-scale modernisation of these networks is desired.

6. Tolls are generally more appropriate in countries where road-user taxes are not high. In any case, they should be limited to special instances of providing important motor roads or self-contained major projects where the benefit to the user obtained by paying the toll is clearly greater than the toll rate paid.

7. The financing of international roads should be primarily the duty of the respective countries crossed by the roads. If financial aid through international agencies becomes possible, international benefits from these roads as determind by traffic census could be a basis for such an aid. In any system adopted one should be inspired by a feeling of international solidarity.

(b) Economic Justification of Road Works

1. It is observed that, in general, fiscal quarters of governments have not yet been enlightened about the favourable effects on the overall national economy of investment to imrpove motor transport. Further economic research on the subject and convincing calculations are therefore necessary.

2. A satisfactory method of analysing the expected economic benefits of road construction to an under-developed region has yet to be found.

3. One way towards a method of determining the justification of road works should be suggested as an estimation of the contribution of these works to the national income.

4. Further experiments and studies are necessary to compile facts concerning the effect on costs of road maintenance and vehicle operation of varying kinds of improvement. Such data should be available for those responsible for road planning so that they can be used in calculating the economic justification of improvements.

5. Experience in Europe seems to confirm the observations in America that accidents per million vehicle-kilometre on limited access motor roads are considerably less than half of the accidents on other roads.

SECTIONS 1 AND 2 COMBINED

Sixth Question : Urban Roads and Traffic in Cities

(a) General

1. The methods adopted to deal with traffic in cities should be based upon the long-term master plan.

Studies dealing with traffic should take into account the estimates of future volumes.

In these studies the width of roads should be calculated on the basis of apporximately 3 metres for each traffic lane.

2. These following conclusions are intended to be general recommendations—where dimensions and figures are given they represent a reasonable minimum or maximum, but will need to be amended to suit special circumstances.

3. The longitudinal gradient should not exceed 10 per cent, as a maximum and not be less than 0.5 per cent as a minimum.

4. The ratio of maximum rise to the width of road should be, generally :

Plain macadam roads	...	1/50
Irregular stone roads	...	1/60
Bituminous surfacing	...	1/60
Stone sett paving	...	1/65
Bituminous pavings	...	1/70

In cities that have a suitable street cleaning department and more advanced drainage systems, these figures could be reduced.

When plane surfaces are used for the haunches, the slope can be reduced to less than 2 per cent.

5. Because of the disadvantages resulting from mains of all kinds under the subsoil, placing of conduits under carriageways should be avoided.

Even if this principle is applied only to telephone, electricity, gas and water mains, a minimum width of at least 3.50 metres should be available.

If trees are to be planted, footways should have an additional width of 2 metres.

It results from these considerations that the minimum width for future road designs is 3.5 metres for footway, 8 metres for the carriageway, or a total of 15 metres between boundaries.

6. Attention should be drawn to the choice of surfacing for urban roads. The main points to be considered are as follows:

(a) The frequent stresses of braking and acceleration resulting in a certain fatigue of the surfacing.

(b) Stationary vehicles leave patches of oil or fuel which may cause damage to some surfacings.

(c) Machines with roller-brushes for cleansing may cause appreciable wear.

(d) The spreading of salt to induce the melting of snow may cause damage to some surfacings.

(e) Drainage water containing humic matter may cause damage to some surfacings.

(f) In certain districts the surfacing used should have sound deadening properties.

7. On a road with tramway lines a good practice is the use of sett paving with mortar joints.

8. For surfaced footways a slope of 2 per cent is a minimum for the transverse profile.

(b) Car Parks and Parking

1. Reasonable parking facilities must be provided in cities with heavy traffic movement. First of all public or private off-street parking spaces should be encouraged so that the streets should not be used for parking. This includes :

(a) Wide, open-spaced verges ;

(b) Roads whose width is greater than that required by traffic ;

(c) Creating parking lanes on footways of wide thoroughfares ;

(d) Covering railway cuttings ;

(e) Setting aside certain places on the periphery for providing parking places ;

(f) Making use of the banks of the rivers ;

(g) Use of special garages, constructed below public grounds.

2. The master plan should include underground car parks, in some cases under the buildings, but generally under the open spaces left between the buildings.

3. When a new street is planned, from the very start of its building, convenient parking spaces should be foreseen.

The planning authorities should generally require that new buildings shall contribute to the solution of the parking problem by providing accommodation within their sites.

For this purpose a study should be made of the extent to which buildings generate such a parking demand in respect to their type and sizes. Values should be based on increased demands for parking in the near future.

4. It is desirable that places of public assembly such as railroad station squares are provided with parking spaces for buses and taxicabs.

5. Unilateral parking on alternate periods should be used on throughfares with carriageways having less than four-vehicle lanes.

6. The desire to park near the points where the inhabitants are going is understandable. It follows that the selection of a site for a parking place should be made very carefully.

7. Parking restrictions, including time limits and prohibition, should be enforced by the authorities for the congested central areas.

8. It is suggested that timing systems should be used to enforce a time-limit on the authorised parking places on the streets in the central business section.

(c) Layout and Equipment of Road Intersections

1. Before choosing a solution to avoid bottlenecks an analysis of the intersection should be carried out to determine the density of total traffic on each carriageway and vehicle lane, proportion of different vehicle types, percentage of vehicles turning left (contrary in countries with left-hand drive), density of pedestrian traffic, number of accidents, location of the intersection and the future master plan of cities.

2. The following measures should be considered for the improvement of heavily-trafficked intersections :

(a) Extra wide carriageways should be provided for a distance of about 50 metres at the approaches to intersections.

(b) The alignment of the buildings near the intersections should be modified for the provision of wide corners.

(c) Prohibiting parking on the approaches to intersections and pedestrian crossings.

(d) The siting of bus stops should be studied carefully.

(e) Sufficient space should be provided for left (contrary in countries with left-hand rule) turning vehicles by means of islands.

(f) Kerb alignments should be provided to permit a vehicle to turn right (contrary in countries with left-hand rule) within the path prescribed by traffic regulations.

(g) Through traffic should be removed to by-pass roads.

3. The width of a draw-in should preferably be 3 metres to maintain the effective width of the carriageway.

Spaces for buses should be reserved in accordance with the number of services and frequency.

4. The turning radius of vehicles should be taken into consideration when designing the layout of kerb intersections.

5. Bad or wrong siting of filling stations is detrimental to traffic and street capacity and thus their locations should be selected with care.

6. Pedal cyclist traffic is the greatest source of difficulty in respect of traffic flow and should be taken into account when calculating requirements of intersections. Cycle tracks are particularly advisable in districts where their construction is justified by the volume of this special traffic.

7. The construction of a roundabout is only practicable whenever :

(a) There is heavy traffic on the converging roads.

(b) The central part is of sufficient diameter.

(c) Vehicle speeds in the weaving arms should be approximately equal.

(d) The weaving angles should be small.

(e) Good visibility should be available from the approaches.

8. Grade separation must be attained if the traffic congestion at intersections is to be rooted out.

But because of their high cost, construction of separated level crossings demands that one should have a sure forecast as to its efficient use in both the near and distant future.

9. The main characteristic of a signal installation is its visibility and this necessitates a detailed study of siting and luminance.

To prevent the sun's rays from shining directly on the lights, these should be equipped with sufficiently large vizors. To prevent dazzle at night some intersections should be equipped with a volt-reducing system.

11. The rear face of traffic signals at multi-phase junctions should be blacked out, if too many untimely starts are to be avoided.

12. If intersections are placed fairly close to each other the system should be co-ordinated, this increases traffic speed.

13. The flexibility of vehicle-actuated control enables it to be applied efficiently to many types of intersections and every class of traffic with the minimum of traffic delay consistent with safety.

Some semi-vehicle-actuated installations should also be used, having detectors in the side roads only. This idea should be accepted for holding up vehicular traffic elsewhere than at a junction while pedestrians cross roads.

14. Both fixed-time and vehicle-actuated controls should be used in linked systems varying from the simplest of links between the adjacent two junctions to co-ordinated flexible progressive systems covering many junctions.

15. The largest capacity at a signalised intersection is obtained when the number of phases is a minimum, i.e., two.

The timing employed depends on the junction layout and traffic density, but the total cycle time is approximately 45 to 70 seconds for two-phase control.

Generally a control device should enable the total length of a cycle to be modified automatically according to the time of day.

16. Overlapped (red-amber) three-phase indication should be employed. The length of amber period should remain constant and should be the same for all installations.

To clear the intersections it is essential that the green light on one road should not coincide with the red light which is stopping traffic on the side road. The "all red period" usually is one second, but may be increased, if necessary.

17. There are quite a number of combinations of signals. Standardisation of all these various forms would surely be an advantage to all concerned. This standardisation should not be restricted to the individual country, but made on an international level.

18. Traffic-sign lighting should be provided by external lanterns which are designed to give uniform illumination over the whole face of the sign.

19. Lane markings should be used at the approaches to junctions, particularly signalled ones, to get drivers into the lane more suitable for their subsequent movements.

The marking known as the "Zebra" on pedestrian crossings should have a special significance for right of priority.

The marking on the roadways and other indications for crossings by pedestrians should be different from those indicating stop.

20. The number of signals, signs and markings should be kept to a minimum.

21. Footpaths should be provided at crossings.

Subways should be constructed at sites where both vehicle and pedestrian traffic is very heavy. Signals for pedestrians away from junctions are justified only when pedestrian traffic is heavy throughout the working day.

22. As soon as a carriageway is of a certain width (approximately more than 14 metres) an island with bollard or a central refuge greatly facilitates the pedestrian task of crossing the road, and to some extent increases his safety.

(d) **Tramway Tracks in Built-up Areas**

1. The disadvantages resulting from the use of tramways are mainly attributed to the rigidity of the layout. In some countries there is a marked tendency towards the abolition of tramways. Nevertheless, new tracks are being laid down in some other countries. A thorough survey and investigation of the problem is necessary and the subject should be further discussed in future Congress.

2. Whilst the present tendency is to give preference to vehicles on pneumatic tyres rather than to tramways, the value of the latter traffic should not be overlooked.

3. The removal of tracks is not always possible when a tramway line ceases to be used. The remaining grooved rails are a source of danger, particularly to cyclists. The tracks should be filled either by chippings on to which bitumen emulsion is then spread or with chippings precoated at a mixing plant.

4. Repair of tramway tracks should be made without breaking-up operations.

5. If a distinction is to be made between public transport, the former should be given preference.

(e) **Waterproofing and Drainage of Carriageways**

1. Since water has such a pernicious effect on the subsoil, deforms the substructure, and damages the pavement, the water should be prevented from penetrating into the subsoil and means provided for draining it off as quickly as possible.

Rapid drainage of rainwater from the town streets is obtained by adoption of crossfalls, construction of pavements which are suitably raised according to the type of surfacing used, and setting out the drains at predetermined points and suitable spacings.

2. In places where there are gentle longitudinal gradients, special type of kerbs should be used. On wide roads with steep

gradients it might be of advantage to use for the purpose of catching surface-water the special " Nery " type inlets.

It appears impossible to drain the water with a gradient less than 5 millimetres per metre: with this gradient, the inlets should be placed at a maximum spacing of 50 metres.

3. The kerbstones should be made from limestone or granite and generally should be about 10 to 15 centimetres above the water in the gutter.

4. A gutter inlet should be placed at each suitable point to conduct the surface and cleansing water to the used-water system. Manholes should be arranged at all crossings, at the points where sewers change direction or size or gradient, and in the straight sections so that there is a distance of not more than 60 metres between two successive manholes.

5. The wearing course itself is usually waterproof and this is especially so in the case of bituminous or concrete surfacings.

6. There should be better means of preventing surface water from penetrating between rails and carriageway surfacings. The intermediate joint should be 15 to 50 millimetres and filled with a suitable compound.

7. In order to lower the phreatic level and drain off free (gravitational) water, drains should be used.

8. The opening of trenches should be avoided as far as possible by taking the following measures:

(a) Preparing a programme of street construction or repair and sending their programme to the town authorities.

(b) Prohibiting the supply companies from opening trenches in roads where surfacing has been renewed during the preceding 5 years, except in case of urgent necessity.

(c) Installing a sufficient number of pipes of appropriate dimensions to permit the passage of mains, especially at intersections, during the construction of roads.

(f) Ribbon Development

1. This is a problem that most European countries are considering and trying to solve. Our inclination, however, is to leave this problem to the next Congress so that more attention and time can be devoted to it.

RESEARCH SECTION

EFFECT OF SIZE ON THE COMPRESSIVE STRENGTH OF SOIL BLOCKS

By

H. L. Uppal

[This Research Note was received in the office of the Indian Road Congress on 15-1-1955. Following precedent, it was circulated to the Member of the Research Organization Division, and comments invited by correspon dence. The comments received were then sent to the Author for his reply The Note along with the comments and replies of the Author was discusse at the Research Organization Division meeting held at Delhi on *12-12-1956* It is now published along with the comments and directions of the Commit tee for further work for information of Members.]

1. INTRODUCTION

1.1. In the study of soils for engineering constructions, the test for compressive strength is an important one. Yet different sizes of test specimens have been used.

1.2. Research has revealed that the strength of a concrete mix varies with the shape and size of the specimen tested. It has been shown[1] that the height of the specimen in relation to its width greatly influences the results. It has also been proved that the size of the specimen affects the compressive strength, smaller the cube greater the strength[2].

1.3. Different types of specimens have been tried for testing cement-soil mixtures. Amongst the cube specimen, 3-in. cube has been frequently mentioned[3], but Maclean[4] and others used 4 in. cubes. The Portland Cement Association[5] of America has recommended 2 in. by 2 in. cylindrical blocks, whereas Clare and Packham[6] used cylindrical moulds of 2 in. diameter and 3 in. height. It has been the practice to use cube specimens in England and cylindrical moulds in U. S. A. Recent trend has, however, shifted in favour of cylindrical moulds with height twice the diameter[7] to ensure a plane of shear, varying between 45 and 60 degrees. The same has also been adopted as a British Standard[8].

1.4. No information regarding the size of the block for testing the compressive strength of soil alone is available. Unlike cement-soil mixtures, on account of low compressive strength of some of the soils, it is not always convenient to make blocks of double the diameter in height. Also, in the laboratory both for routine and research purposes it is not desirable to deal with either too long or too big blocks. It is, therefore, necessary to know

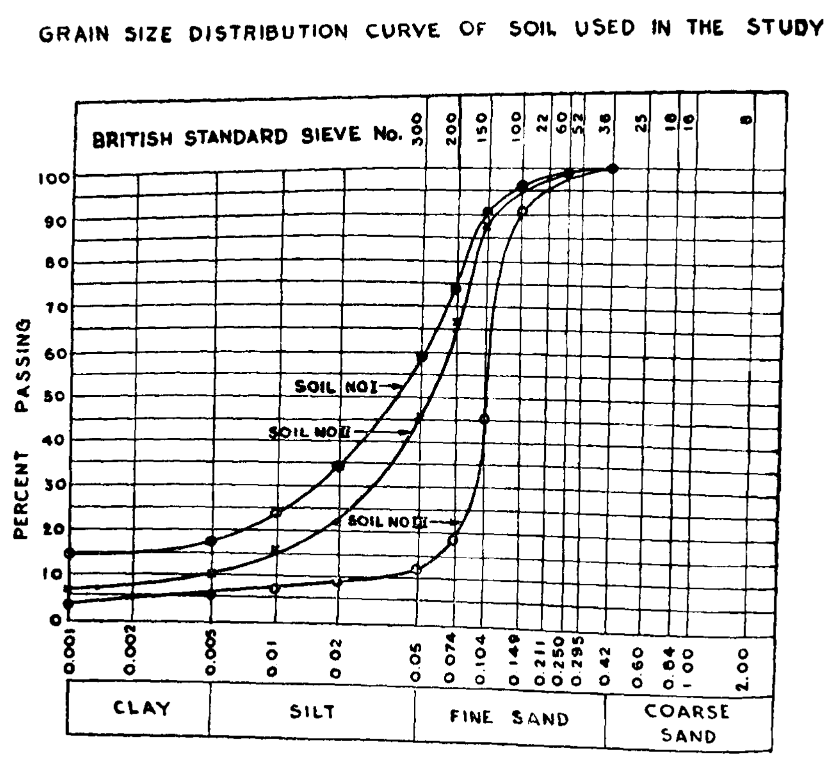


Fig. 1. **Grain size distribution curve of soils used in the study.**

how the height affects the compressive strength of a soil block. It is also desirable to know how the strength obtained with one kind of test specimens differs from that obtained with others. With this object in view the study has been conducted with some of the soils locally available.

2. SOILS SELECTED

Three alluvial soils of varying plasticity indices met with in Delhi were selected for the study. The detailed analysis of the three soils are given in Table 1, page 142, and the grain size distribution curves are shown in Fig. 1.

3. DRY BULK DENSITY OF THE TEST PIECES

To find out how the dry bulk density affects the compressive strength with different height-diameter ratios, specimens with dry bulk densities of 1.6, 1.7 and 1.8 were made. These densities were chosen because they were generally adopted for different types of construction. It was found that soil No. III which is predominent in fine sand, could not be compacted in the laboratory under conditions similar to those of other soils to a density of 1.8. The result for this soil, therefore, is given for densities of 1.6 and 1.7 only.

4. SIZE OF THE TEST SPECIMEN

4.1. In most of the research laboratories in India modified Abbot's cylinder[9, 10], Photo 1, is commonly used for determining optimum moisture and consequently moulding of blocks for laboratory investigation. It was, therefore, selected for compacting purposes.

4.2. Blocks with 1, 2, 3, 4, 5 and 6 in. heights and different densities were made. Different densities were obtained by varying the number of blows of the hammer. The blocks were dried at 50°C. to constant weight after curing for 7 days and then subjected to compression test, Photos 2 and 3. The results obtained are given in Table 2, page 143, and shown in Fig. 2.

5. STUDY WITH CEMENT-SOIL MIXTURES

To find out how the addition of cement will change the compressive strength of blocks with various height to diameter ratios, blocks of 5 per cent cement-soil mixtures with densities of 1.6, 1.7 and 1.8 were made using the three soils. The compacted blocks were cured under wet sand for a week and then allowed to dry to a constant weight at about 50°C. before subjecting them to test. The results are given in Table 3, page 144 and shown in Fig. 3.

6. STANDARD SHAPES AND SIZES OTHER THAN ABBOT'S

6.1. For finding out the compressive strength of cement-soil mixtures, various workers have used different shapes of the specimen. Important amongst these are :

(a) Proctor's[11] mould 4 in. diameter and 4.59 in. high

(b) 4 in. cubes ; and

(c) 3 in. cubes.

With the object of finding out how the values of compressive strengths of blocks having the above mentioned shapes and sizes compare with those of various heights using Abbot's cylinder, specimens of all the three soils were made having densities of 1.6, 1.7 and 1.8. These blocks which are bigger than the ones made using Abbot's cylinder were slowly dried (to avoid cracks) to constant weight. The results of compression tests are given in Table 4, page 145 and plotted in Fig. 2.

7. DETERMINATION OF THE COMPRESSIVE STRENGTH

The compressive strength of soil blocks of varying sizes was determined after they had attained constant weight at 50° C. The drying time differed with the size of the block, and was as much as 100 hours for the bigger sizes. The compression test was carried with K.C. Productions Loading Frame No. 131 fitted with a proving ring.

8. THE RATE OF LOADING

The rate of loading generally specified for soil-cement mixtures was 250 lb per sq. in. per minute. This was considered to be too high for the low compressive strength of soils. The rate of loading was, therefore, reduced to 125 lb per sq. in. per minute.

Three blocks of each specimen were tested.

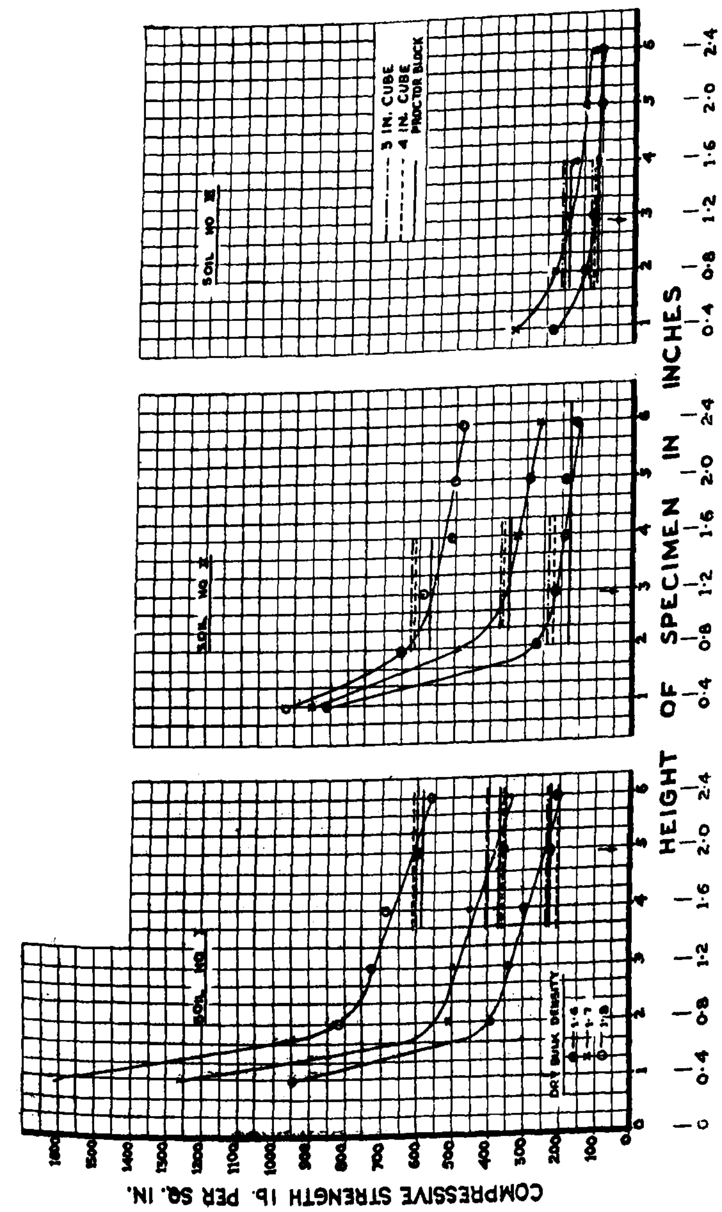


Fig. 2. Compressive Strengths of different sizes and shapes of specimens compacted to different densities.

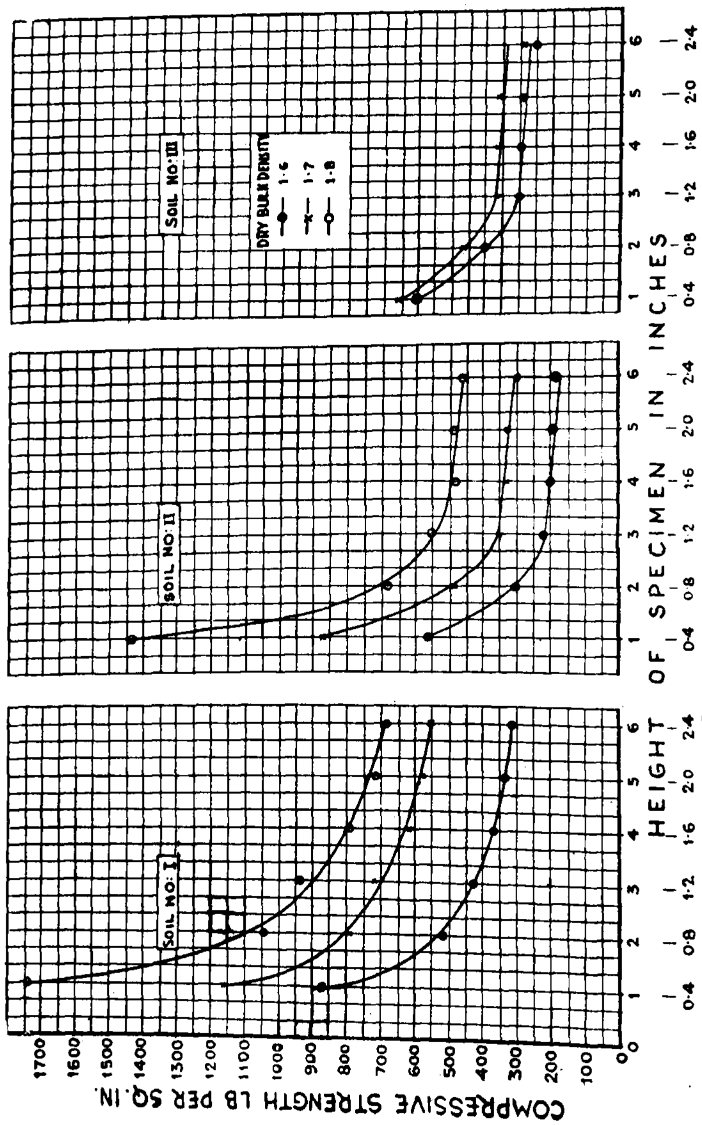


Fig. 3. Compressive strengths of cylindrical cement soil blocks, diameter 2.5 in. with different heights and densities.

9. DISCUSSION OF RESULTS

9.1. The results given in Tables 2 and 3 show:

(a) That the compressive strength of the cylindrical block is reduced as the height of the block increases. There is, however, a sudden fall in compressive strength as the height is increased from 1 to 2 in., but with further increase in height up to 6 in., the reduction in strength is uniform. This trend is similar to the change in compression strength of cylindrical concrete blocks of increasing height.

(b) For the same soil, the rate of fall in compressive strength with increasing height of the test specimen is not affected by the degree of compaction, except that lower densities give lower values.

(c) The values for a height-diameter ratio of 0.8 and above, fall on a straight line with a certain slope. The slope of the curve relating the compressive strength to the height-diameter ratio is almost the same for different dry bulk densities of the same soil; but it decreases as the sand content of the soil increases. Once the slope for a given soil is known, it would be possible to co-relate the values of compressive strength when blocks of different heights are prepared using Abbot's cylinder.

(d) The abnormal high compressive strengths obtained for blocks with a height-diameter ratio of 0.4 may be ascribed to the influence of horizontal shearing stresses which are developed at the interfaces of the specimen when subjected to load. For bituminous pavements, this particular aspect of the problem has been discussed by Smith[12]. He has shown that rough specimens develop high coefficient of friction while in "rich" mixes, friction is reduced due to lubricating properties of the thick asphaltic binder films. It was suggested that rubber pads on top and bottom of the specimen would reduce friction.

9.2. When testing the 1-in. high block ($h/D=0.4$), attempts were made to reduce the end friction by providing at the top and bottom of the specimens the following:

(i) foamed rubber pieces $\frac{1}{2}$ in. thick,

(ii) greased rubber pads $\frac{1}{4}$ in. thick, and

(iii) greased rubber pads $\frac{1}{8}$ in. thick.

It was observed that with the use of the pads mentioned above the load could not be properly distributed on the specimen. There was a tendency for the load to act as a point load, resulting in the splitting of the block vertically. The values obtained were even lower than those obtained with specimens of 6-in. height.

9.3. **Soil-Cement Mixtures.** The data given in Table 3, page 144 confirm the observation made earlier that blocks of 1-in. height give abnormally high values when compared with the values obtained for other heights. The addition of cement to soil reduces the difference in compressive strength of 1 and 2 in. high blocks irrespective of the type of soil used or the density to which the specimens were compacted.

Unlike when soil alone is used, when cement is added to clayey soils the compressive strength of the blocks with 2 to 6 in. height does not have a straight line relationship with the height-diameter ratios. In cement-sandy soil mixtures, the effect of varying the height on the compressive strength is less and it is noticed that blocks with 3 to 6 in. height have practically the same compressive strength.

10. TEST WITH STANDARD SHAPES AND SIZES OTHER THAN ABBOT'S

The results in Table 4, page 145, show that the compressive strengths obtained with Proctor's mould, 4-in. cubes and 3-in. cubes are almost the same. The compressive strengths of 4-in. cubes are slightly higher than Proctor's mould and those of 3 in. cubes slightly higher than 4 in. cubes, Photos 4 and 5. This is true of all the soils tested and all the densities employed.

11. CO-RELATION OF COMPRESSIVE STRENGTH OF DIFFERENT SHAPES OF TEST SPECIMENS

11.1. Fig. 2 shows that in soil No. II and III the compressive strength obtained by using specimens of various shapes and sizes, at all densities, is more or less the same as the values obtained with 3-in. high cylindrical blocks of the Abbot's apparatus. This is true only for those soils which are not very clayey.

11.2. A 3-in. high block with 2.5 in. diameter is very convenient to handle, and can easily be compacted in two layers in Abbot's cylinder. It is, therefore, proposed to adopt this size for compressive strength of blocks, while working with Abbot's

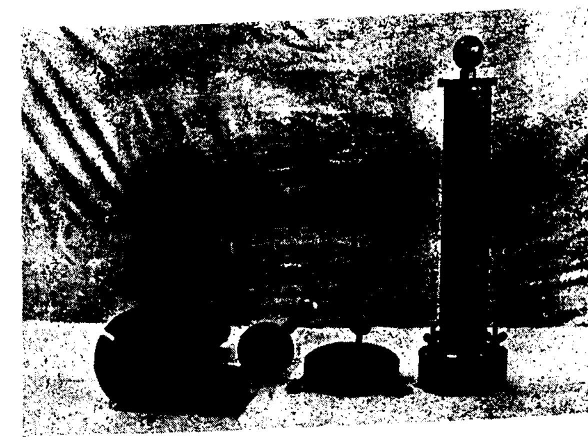


Photo 1.

Modified Abbot's cylinder.

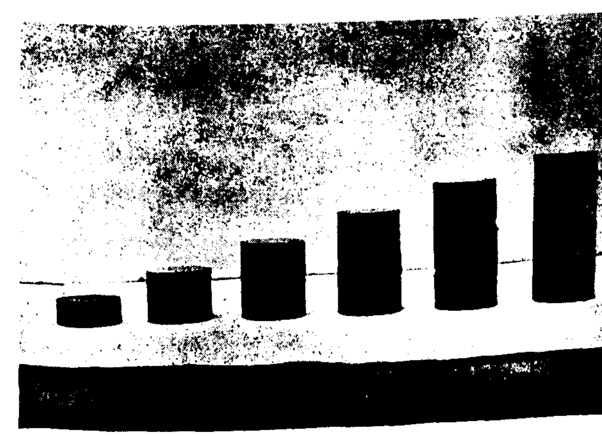


Photo 2.

Soil blocks of various heights from Abbot's Cylinder (untested).

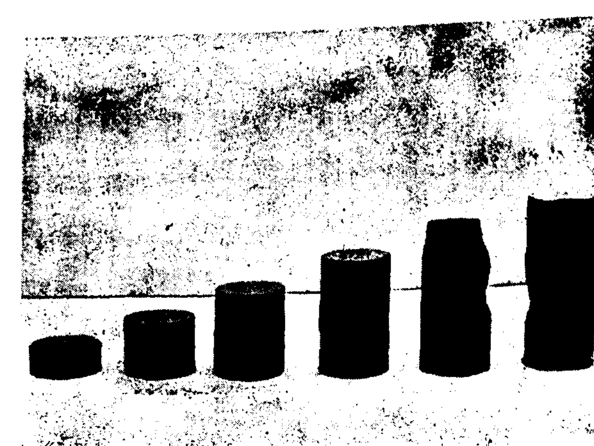


Photo 3.

Soil blocks of various heights from Abbot's cylinder (tested).

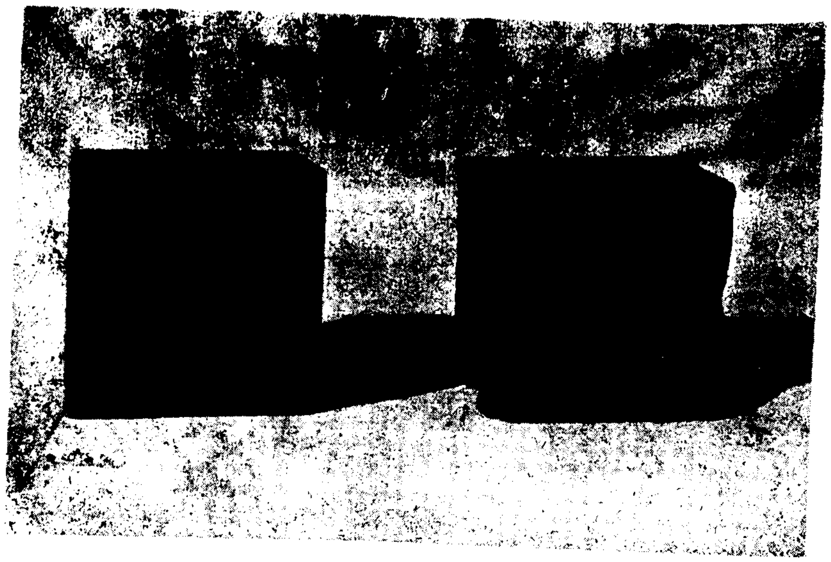


Photo 4.
4 in. soil cubes – uncrushed and crushed.

Photo 5.
Soil blocks from Proctor's mould – uncrushed and crushed.

cylinder, as it gives comparable values with other standard test specimens.

11.3. For clayey soils which have enough cohesion and consequently high compressive strength such as soil No. I, it is noticed that the compressive strength obtained by using various sizes and shapes of the specimens compacted to densities of 1.6, 1.7 and 1.8 is practically the same as the values obtained with Abbot's cylinder of 5-in. height, which shows that the theory of having twice the height than diameter in cylindrical blocks for purposes of compressive strength would apply to rigid material and soils which are fairly clayey.

11.4. If for purposes of uniformity, 3-in. high block is used for convenience sake, then the values should be considered high by 120 to 150 lb per sq. in.

11.5. It will be noticed that there is big gap, in the plasticity indices of the soils selected. So, it is not possible to foresee how far the values with 3-in. cylindrical block will compare with the values obtained from other standard specimens. The specifications generally prescribed for stabilized soil works are—liquid limit not more than 26 and plasticity index between 9-10. A soil conforming to these sqecifications was, therefore, selected and blocks of the different test specimens with densities 1.6, 1.7 and 1.8 were made. The results are given in Table 5.

TABLE 5

Compressive Strength of Test Specimens

Soil characteristics	Description of test specimens	Density in gm per cu.cm.	Compressive strength (crushing) lb per sq. in.			
			1	2	3	Average
Liquid limit 24.7	3 in. high Abbot's cylinder	1.6	190.6	166.2	176.0	177.6
		1.7	290.0	300.0	285.0	291.7
		1.8	437.9	437.9	437.9	437.9
Plasticity index 9.3	3 in. cubes	1.6	213.4	190.9	213.4	205.9
		1.7	281.4	323.4	294.3	299.7
		1.8	407.5	420.4	447.1	425.0
Fraction coarser than 200 sieve 46.2	Proctor's mould	1.6	122.0	107.0	102.0	110.3
		1.7	275.0	270.0	—	272.5
		1.8	480.0	383.0	347.0	403.3

It will be seen from the above results that 3-in. high Abbot's cylinder blocks give fairly comparable values of compressive strength with those of other test specimens.

12. SUMMARY

The compressive strength of a soil block depends upon its size and shape. When cylinders of different heights with a diameter of 2.5 in. are used, the strength falls as the height of the sample increases. The compressive strengths obtained with cylindrical blocks 3 in. high and 2.5 in. in diameter are practically the same as those obtained with 3 in. cubes, 4 in. cubes and Proctor's moulds. This is true for all the soils tested (with a maximum plasticity index of 10) and the different densities employed. It is suggested that the height of the test specimen for compressive strength test be 3 in. when the Abbot's modified cylinder is used for compaction.

ACKNOWLEDGMENT

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TABLE 1

Physico-chemical Characteristics of Soils selected for Study

		I Clay Soil	II silty soil	III sandy soil
Liquid limit		33.37	25.56	24.00
Plasticity index		14.00	6.16	3.08
Fraction coarser than 200 mesh B.S.S.	Per cent	26.02	33.78	80.84
Silt+clay passing 300 mesh B.S.S.	Per cent	59.80	46.74	12.26
Total *Soluble salts*				
Solid ...	Per cent	0.074	0.077	0.022
Sulphates ...	Per cent	Nil	Nil	Nil
Chlorides		0.017	0.012	0.016
Exchangeable : +*Ca* per 100 gm of soil*		12.98	3.90	3.7
Exchangeable Na+*K* per 100 gm of soil*		0.5	0.8	Trace
$CuCO_3$ ...	Per cent	8.0	0.4	Nil
MECHANICAL ANALYSIS				
Percentage of fraction passing British Standard Sieve No.	36	100.00	100.00	100.00
	60	97.82	97.36	98.50
	100	95.74	95.84	90.10
	150	90.02	88.66	45.50
	200	73.98	66.22	19.16
Per cent smaller in size than	0.05 mm	59.80	46.74	12.26
	0.02 mm	34.23	22.60	9.36
	0.01 mm	23.84	15.72	7.56
	0.005 mm	16.57	11.53	6.17
	0.001 mm	14.63	8.64	4.03

* In milli equivalents.

TABLE 2

Compressive Strengths of Cylindrical Soil Blocks—Diameter 2.5 in. with different Heights & Densities

Dry density gm. per cc	Height of block in inches	COMPRESSIVE STRENGTH IN LB PER SQ. IN.											
		SOIL No. I				SOIL No. II				SOIL No. III			
		1	2	3	Average	1	2	3	Average	1	2	3	Average
1.6	1	1042.8	899.3	899.3	947.1	875.7	891.0	819.7	862.1	219.9	244.3	244.3	236.3
	2	397.1	381.8	397.1	392.0	269.8	264.7	269.8	268.1	151.5	146.6	141.7	146.6
	3	336.0	330.9	356.4	341.0	259.6	195.5	190.6	215.2	107.5	122.1	131.9	120.5
	4	305.4	285.1	290.2	293.6	190.6	190.6	200.4	193.9	112.4	107.5	102.6	107.5
	5	229.7	224.8	229.7	228.1	185.6	185.6	205.2	192.1	107.8	102.9	—	105.3
	6	200.3	205.2	—	202.7	146.6	156.4	166.1	156.4	112.7	107.8	93.1	104.5
1.7	1	1394.7	1246.3	1134.1	1264.3	891.0	912.4	899.3	900.9	356.4	356.4	346.2	353.0
	2	519.3	498.9	504.0	507.4	437.8	565.1	478.6	493.9	244.3	234.6	224.8	234.6
	3	458.2	529.5	519.3	502.3	356.4	346.2	366.5	356.4	185.7	176.5	224.8	195.7
	4	448.0	448.0	458.2	451.4	341.1	300.4	346.2	329.2	185.7	171.6	176.5	177.9
	5	366.5	351.3	351.3	356.4	285.1	290.2	295.3	290.2	137.2	147.0	147.0	143.4
	6	341.1	346.2	341.1	342.8	264.7	269.8	269.8	268.1	132.3	127.4	107.8	122.5
1.8	1	1603.2	1551.1	1681.4	1611.9	1029.7	958.0	938.4	975.3				
	2	840.1	794.2	24.8	819.7	641.5	651.7	651.7	648.3				
	3	712.8	717.9	748.4	726.3	610.9	575.3	—	593.1				
	4	712.8	661.9	—	687.3	509.1	509.1	—	509.1				
	5	590.6	590.6	600.8	594.0	509.1	483.7	509.1	500.8				
	6	570.2	565.1	—	567.6	493.8	478.6	483.7	483.7				

TABLE 3

Compressive Strengths of Cylindrical Cement-soil Blocks—Diameter 2.5 in. with different Heights & Densities

Dry bulk density	Height of block in inches	COMPRESSIVE STRENGTH IN LB PER SQ. IN.											
		SOIL No. I				SOIL No. II				SOIL No. III			
		1	2	3	Average	1	2	3	Average	1	2	3	Average
1.6	1	906.3	835.0	—	870.7	565.1	554.9	—	560.0	600.8	616.1	—	608.5
	2	534.6	534.6	483.7	517.6	325.8	305.5	395.3	308.8	387.0	407.3	—	397.2
	3	446.0	435.8	397.1	426.3	234.6	224.8	200.4	219.9	295.3	290.2	—	292.2
	4	402.2	371.6	325.9	366.6	210.1	185.7	200.4	198.7	285.1	295.3	—	290.2
	5	346.3	341.2	315.7	334.4	205.6	190.7	175.9	190.7	280.0	295.0	280.0	285.0
	6	331.0	315.7	295.7	314.0	180.8	175.9	171.0	175.9	269.8	215.0	—	242.4
1.7	1	1173.1	1147.0	—	1160.1	875.7	865.5	—	870.6	697.7	682.4	621.2	667.1
	2	769.0	808.1	—	788.5	498.9	488.7	458.2	481.9	478.6	451.1	—	464.8
	3	763.7	733.2	662.0	719.6	356.4	346.2	—	351.3	371.6	366.5	361.4	366.5
	4	621.1	600.7	—	610.9	346.2	336.0	315.7	332.6	346.1	320.7	397.1	354.6
	5	610.9	570.2	534.6	571.9	336.0	330.9	320.7	329.2	320.7	397.0	—	358.9
	6	560.0	544.7	539.6	548.1	300.4	295.3	295.3	297.0	310.5	264.7	—	287.6
1.8	1	1772.7	1863.9	1603.2	1746.6	1433.8	1433.8	—	1433.8				
	2	1068.8	977.6	1094.9	1047.1	702.8	677.3	662.0	680.7				
	3	967.4	957.2	875.7	933.4	565.1	549.8	534.5	549.8				
	4	824.8	753.5	773.9	784.1	493.6	493.6	468.2	485.1				
	5	723.0	697.5	—	710.2	488.5	478.4	478.4	481.8				
	6	697.5	672.2	—	684.9	473.3	468.2	458.0	466.5				

TABLE 4

Compressive Strengths with different Densities, Shapes and Sizes of Specimens

Soil No.	Dry bulk density gm per cc	Compressive strength lb per sq. in.			
		1	2	3	Average
		Proctor mould			
I	1.6	260	236	190	229
	1.7	367	367	367	367
	1.8	597	571	—	584
II	1.6	184	178	—	181
	1.7	367	357	311	345
	1.8	592	551	—	571
III	1.6	122	117	102	114
	1.7	204	194	173	190
		4 in. cubes			
I	1.6	220	220	196	205
	1.7	420	340	320	360
	1.8	632	600	600	610
II	1.6	244	232	212	222
	1.7	392	368	334	365
	1.8	608	604	632	615
III	1.6	132	119	114	122
	1.7	232	217	156	202
		3 in. cubes			
I	1.6	236	233	—	235
	1.7	382	420	410	403
	1.8	630	605	580	605
II	1.6	249	229	226	235
	1.6	388	382	329	366
	1.8	626	624	623	624
III	1.6	—	137	118	127
	1.7	—	216	197	207

COMMENTS

Shri K. F. Antia felt that the compression tests were not likely to furnish information of value regarding the strength or any other property of the soil. So, there was no advantage in determining the size of specimen for compression tests.

It was universally recognised that compressive strength increased with the increase in dry bulk density for all materials. That being so, what was the necessity for using different densities for the same soil? The density aimed at in the field was the maximum dry density for the particular compactive effort used.

If the dry density was expressed in C. G. S. units, a difference of 0.1 in the density would not convey the full import of the change. The same difference expressed in lb per cu. ft. would be more than 6 lb, the significance of which could be easily appreciated. It seemed desirable to express the bulk density in lb per cu. ft. in accordance with standard practice.

Proctor Compaction Test was internationally accepted, and there was no reason why a departure from the standard practice should be made in using the Abbot's Cylinder. Following a standard procedure would be of immense value in the matter of comparison of research information, etc. Besides, Abbot's Cylinder could not be used for compacting coarse-grained materials.

The test specimens of cement-soil mixtures should not be dried before testing. The compressive strength of damp specimens gave a more correct and reliable indication regarding the rate of development of strength, their suitability etc. than dry specimens.

Use of end packings in crushing strength tests was a disadvantage, for example, in concrete end packings were known to cause a reduction of 30 to 50 per cent in strength.

Shri P. L. Varma mentioned that if the two layers of the specimen were equally compacted, there would be stratification. During the testing of the block accurate result might not be obtained. It would, therefore, be desirable to give details of the mode of compacting the block to a height of 3 in. in two layers.

AUTHOR'S REPLY

The compressive strength of soil was recongnised as an index of the strength of the soil in the various Papers published in the I. R. C. Journal. Since the tests were carried out on

blocks of different sizes, for want of relationship between strength and size of block, it was not possible to compare the results obtained.

Even though it was true that the compressive strength increased with density, it had not been established that one and the same relationship applied to all degrees of compaction. A soil could give different densities, depending upon the mode of compaction, and the compacting effort available at different places might be different. Also, it was sometimes not economical to go in for the highest density attainable.

The metric system was generally preferred and was, therefore, used.

It was the intention of the study to use small quantities of soil, simple apparatus, and still get equally good results. So, Abbots cylinder was used.

A uniform procedure for testing cement-soil mixtures had to be selected. As suggested by **Shri K. F. Antia,** it would, however, have been ideal to saturate the sample before testing.

The use of rubber pads was advocated for testing bituminous mixtures. Therefore, it was tried in the investigation also.

After the first layer was compacted, the surface was scarified and the second layer compacted. This provided good bond between the two layers and it was found that stratification, as mentioned by **Shri P. L. Varma**, did not occur.

DIRECTIONS OF THE RESEARCH ORGANIZATION DIVISION FOR FUTURE WORK

It was agreed that the tests should be repeated with soils having different Plasticity Indices. The rate of loading and the rate of strain should be taken into consideration and the specimen should be coned both at the top and the bottom. It was generally accepted that the use of pads gave wrong results and that their use should be deprecated.

It was further decided that, in addition to the tests being repeated at the Central Road Research Institute, they should be carried out at other laboratories also. Representatives of laboratories from Karnal, Madras and the Railway Testing and Research Centre, Lucknow offered to undertake the work.

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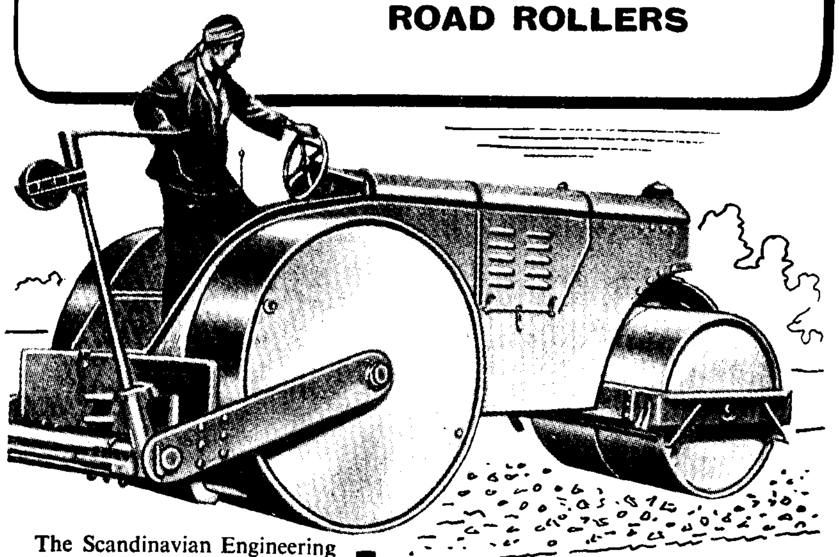

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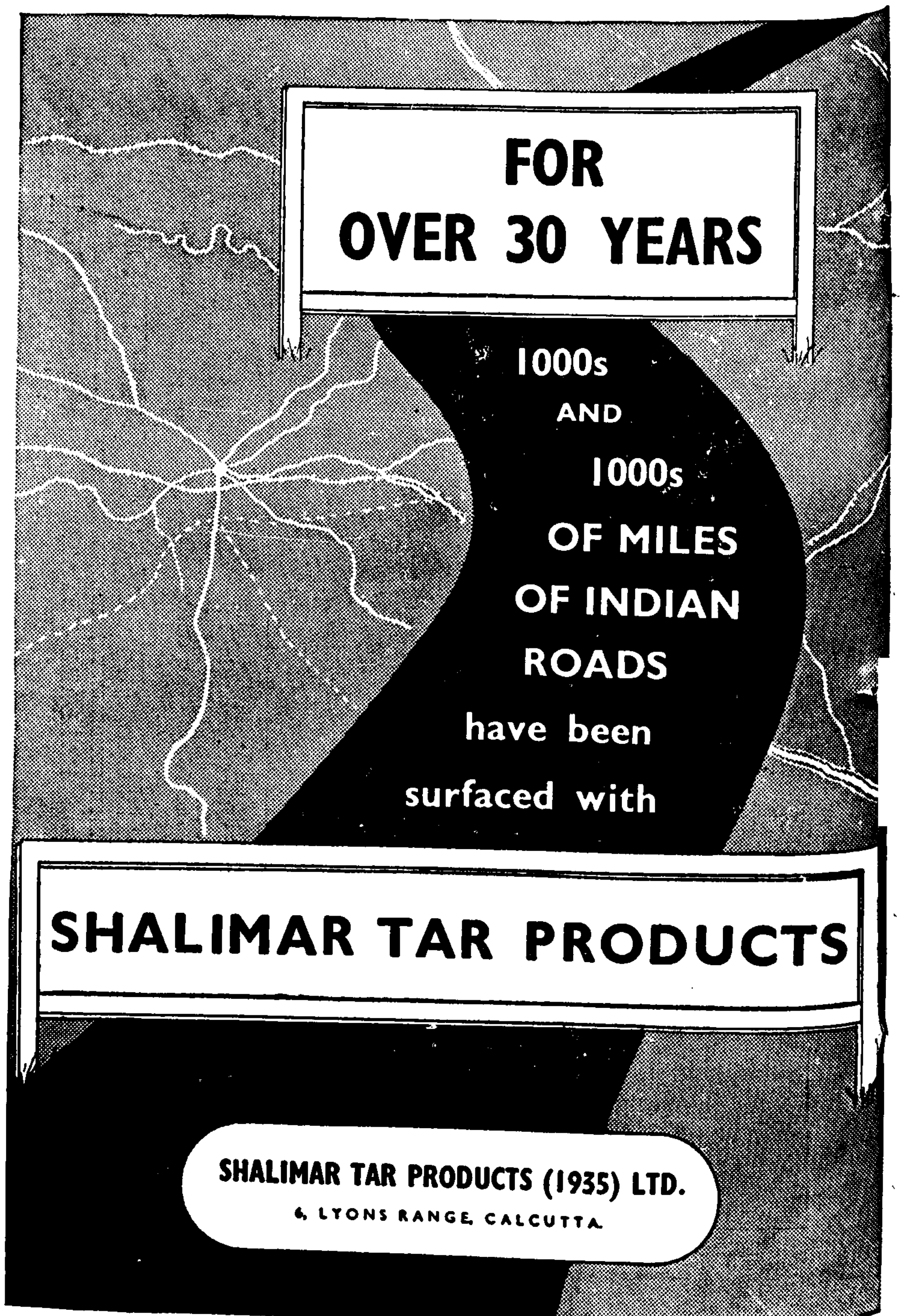
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