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Ahmad N.S.

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INDIAN ROADS CONGRESS

NOTICES AND ANNOUNCEMENTS

NEW ADMISSIONS

LIST OF MEMBERS OF THE INDIAN ROADS CONGRESS ENROLLED FROM 1-8-1955 TO 30-9-1955.

Serial No.	*Roll No.*	*Name.*	*Address.*
1	1961	G. C. Sarmah	Executive Engineer, P.W.D., P. O. Goalpara (Assam).
2	1962	Ganesh Prakash Lall	Assistant Engineer, P.W.D., Chief Engineer's Office, P.W.D., R. & B. Secretariat, Patna (Bihar).
3	1963	Rajinder Singh	Sub-Divisional Officer, Akhnoor, R. & B Division, P. O. Jammu (Tawi).
4	1964	B. V. Gururaj	Junior Engineer (Highways), Bridges, Brahmaur, Via Mangalore.
5	1965	R. B. Desai	Engineer, D.L.B., "Amidhara", Near Char Rasta, Maninagar, Ahmedabad 8.
6	1966	A. R. Mathur	Assistant Engineer, Planning, Sub-Division No. 2, P.W.D., B. & R., Jaipur (Rajasthan).
7	1967	S. P. Sarker	Assistant Engineer, P.W.D., Nowgong, (Assam).
8	1968	F. Kharkongor	Executive Engineer, Aijal Division, Silchar (Assam).
9	1969	D. N. Konwar	Temporary Engineer, Tura Division, P. O. Tura, Garo Hills (Assam).
10	1970	Om Sagar Sahgal	Asstt. Professor in Highways, Punjab Engineering College, Chandigarh.
11	1971	K. K. Sarin	6, Hospital Road, Jaipur (Rajasthan).
12	1972	Kesri Singh Mehta	Asstt. Engineer, Health Circle, P.W.D., C/o. Shri Lehar Singh Mehta, Joint Law Secretary, Rajasthan Sectt., Jaipur (Rajasthan).
13	1973	J. S. Mathur	Asstt. Engineer, P.W.D., Garmukhteshwar, Distt. Meerut (U.P.).

Serial No.	Roll No.	Name.	Address.
14	1974	V. S. Khilnani	Senior Civil Engineer, Rohtas Industries Ltd., P. O. Dalmianagar, (Bihar).
15	1975	S. M. Ghatge	Proprietor, Modern Builders, Kolhapur.
16	1976	Parmanand	Sub-Divisional Officer, P.W.D., Darbhanga Sub-Division, No. 2. Laheria-sarai, Distt. Darbhanga, (B har).
17	1977	P. C. Punnoose	Asstt. Engineer, P.W.D., Trichur, (Travancore-Cochin State).
18	1978	B. K. Sinha	Technical Assistant, Office of the Chief Engineer, P.W.D., B. & R., Patna, (Bihar).
19	1979	Syed Ziauddin Ahmed	Asstt. Engineer, P.W.D., P. O. Rayagada, Dist. Koraput (Orissa).
20	1980	J. V. Prasad	Asstt. Engineer (Highways), Pennar Bridge Division, Nellore (Andhra State).
21	1981	A. N. Bhattacharyya	Sub-Divisional Officer, P.W.D., M. B. Road Sub-Division, P. O. Mawphlang, (K. & J. Hills).
22	1982	T. M. Jalaluddin	Junior Engineer (Highways), Pennar Bridge Works, Villupuram, South Arcot Dist.
23	1983	C. B. Purohit	Asstt. Engineer, P.W.D., B. & R., Pokaran (Rajasthan).
24	1984	A. Nagoji Rao	Asstt. Engineer (Highways), Dorual, Markapur Taluq, Kurnool District, (Andhra State).
25	1985	S. Hanumantha Rao	Asstt. Engineer (Highways), N. H. 5 Diversion Sub-Division, Rajahmundry.
26	1986	B. N. Banker	Sub-Divisional Officer, P.W.D., Gogri-Jamalpur, P. O. Muskipur, Monghyr, (Bihar).
27	1987	Dharmadeo Prasad	Sub-Divisional Officer, Project Sub-Division, P.W.D., Gaya (Bihar).

ASSOCIATE MEMBER

	60	M/s. Pashabhai Patel & Co., Ltd.	Construction House, Ballard Estate, Bombay-1.

II. DEATH

The Council have received with regret intimation of the following death :

Serial No.	Roll No.	Name.	Address.
1	519	I. C. Chacko	Retired Chief Engineer, Travancore State, Thiruvellah.

III. ADDRESSES WANTED

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

Serial No.	Roll No.	Name.	Address as last known.
1	452	Ajwani, J. M.	C/o, Mr. Moti G. Ajwani, No. 56, Bhagwandas Chawl, Bhavani Peth, Poona.
2	1133	Banerji, S. K.	Assistant Executive Engineer, Assam Rail Link Project, Sevoke, D. H. Rly.

Paper No. 181

"THE DESIGN OF AN EXPERIMENT TO ASSESS THE COST OF OPERATION OF MOTOR VEHICLES ON DIFFERENT TYPES OF ROADS"

By

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AND

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SYNOPSIS

The Paper deals with the design of an experiment to evaluate the cost of operation of motor vehicles on different kinds of surfaces of roads.

In view of the large number of factors that have to be taken into account, a factorial experiment is necessary, and the most suitable design of experiment for minimising the cost and supplying the information required within the time available is discussed. In applying the accepted methods of experimental design in this new field, certain unusual difficulties were met with, and the methods adopted to meet them are also discussed.

1. INTRODUCTION

1.1. The science of experimental design is a comparatively new subject. Its methods were mainly developed in the agricultural field though in the exact experimental sciences also the value of proper statistical design is now being recognized.

1.2. This Paper describes the problems confronted in the design, under the application of scientific statistical principles, of an experiment of intimate interest to road engineers. It is hoped that this Paper will give an illustration of the applicability of statistical methods in less explored fields and create a wide interest in the adoption of scientific experimental design.

2. ORIGIN OF THE PROBLEM

2.1. In 1953 the Council of the Indian Roads Congress appointed, on a recommendation from the Transport Advisory Council, a Committee to study the cost of operation of motor vehicles on different types of road surfaces and the economy of road improvement. The Committee were given about a year's time to furnish their report.

2.2. Though the members of the Committee were, because of their experience, well informed about the magnitude of the differences in the cost of operation on various types of surfaces, and though the results of elaborate experiments carried out in western countries on the subject were available, the Committee was of opinion that it was advisable to substantiate the existence and magnitude of the cost differentials with data obtained from operation on Indian roads. Accordingly they decided to collect detailed figures of operation from as many operators as possible, private as well as official. A detailed questionnaire was drafted and circulated. In response, a large volume of data was obtained. Many of the small private operators were, however, not able to supply all the figures required, but some of the larger nationalised undertakings gave a large volume of data in the form required.

2.3. When the data obtained were processed, it became evident that the rich diversity of factual information obtained was more of a hindrance than a help for the purposes of the study. The problems before the Committee required data extracted under controlled conditions. In other words, an experimental approach was essential.

3. ANALYSIS OF THE OPERATORS' DATA

3.1. Table 1 gives the performance in regard to petrol consumption of fifty buses of the same type and make, every one of which had run about one lakh miles.

TABLE 1

Average Miles per Gallon of Petrol Consumed

Region	Road Surface					
	Concrete		Black-Top		Water-bound Macadam	
	No. of buses	Milage per gallon	No. of buses	Milage per gallon	No. of buses	Milage per gallon
Agra	{ 8 { 16	11.95 12.39*			6	10.9
Allahabad	12	12.10	3	12.6*		
Kumaon					5	10.5†

3.2. This Table shows the degree of consistency in the results obtained and gives a sort of measure of the relative merits of the different types and conditions of surface. Statistical tests on the performance of the 20 buses on 'fair' concrete roads in Agra and Allahabad show the observations to be very consistent, that is to say, the patterns of consumption in respect of 'fair' concrete roads at the two places do not differ significantly. The two experiences can, therefore, be combined and treated as one homogeneous group. A similar test cannot be valid for the two observations for water-bound macadam because it is known that one of the observations relates to hilly area and that, in consequence, there are extraneous factors at work.

3.3. Dividing the data, therefore, into five groups, viz., concrete good, concrete fair, black-top good, water-bound macadam fair, and water-bound macadam (hilly tracts), an analysis

* Asterisked figures relate to "Good Condition" roads while the other figures relate to "Fair Condition" roads.

† Hilly roads.

of variance gives the following results :

TABLE 2

Analysis of Variance

Source of variation	Degrees of Freedom	Sum of Squares	Mean Sum of Squares	F	F for 5 per cent Probability level	F for 1 per cent Probability level
Between groups	4	21.93	5.48	11.57	2.61	3.83
Within groups	45	21.33	0.47			
Total	49	43.26				

3.4. For those not acquainted with the technique of variance analysis, a little explanation may be useful. There are in all fifty distinct observations, grouped into five groups. Each group has a central value represented by its mean, and a spread around its central value which can be represented by its standard deviation or variance. The variance is nothing but the mean value of the square of the deviations from the central value and the standard deviation is the square root of the variance. In regard to each group, the variance worked out is what may be called the group variance and it gives an indication of the extent to which individual values within the group vary among themselves. In other words, it is a measure of the errors due to uncontrolled operating influences when observations are taken. Since this error cannot vary as between one group and another, we can pool the group variances of the five groups (suitably weighted) to give a measure of the uncontrollable error existing in the experiment. Actually, we do not take the weighted mean of the variances but the total sums of squares divided by a number designated as the degrees of freedom which, in this case, is the number of comparisons available, namely, 45.

3.5. We have five groups and a comparison between them is required to ascertain the relative merits of the different surface categories. But, at the same time, these differences themselves are affected by the uncontrollable errors, and the question is whether the differences are large enough to warrant confidence that they could not have arisen from uncontrollable errors only.

Treating the five values as five sets of observations and allowing for the fact that each of the means is derived from a number of observations, we get another pooled sum of squares which divided by the number of degrees of freedom (which is less by unity than the number of observations) gives a mean sum of squares.

3.6. The two mean squares are 5.48 and 0.47 respectively; the former is 11.57 times the latter. If only uncontrollable errors were causing the variations, the probability of the ratio exceeding 2.61 is only 0.05 and the probability of the ratio exceeding 3.83 is only 0.01. Thus it is in the highest degree unlikely that a ratio of 11.57 would arise from random errors only and we can confidently assume the existence of influences differentiating the groups.

3.7. This analysis compares all the five groups simultaneously, but it does not say anything regarding the magnitude of individual effects. Nor does it say that, as between any two particular groups, there is a real differentiation. It might conceivably happen that while as between two particular groups there is no significant difference, there are very marked effects as between other groups. In order to examine this, paired comparisons are necessary. Table 3 gives the figures appropriate for a 't' test between any two of the five groups :

TABLE 3

a denotes Concrete Good. *b* Concrete fair. *c* Black-top good. *d* Water-bound Macadam Fair. *e* Water-bound Macadam Hilly.

Groups compared	Difference of means (miles per gallon)	Value of 't'	Probability level of 't' on 45 degrees of freedom (Per cent)
ab	0.34	1.47	15.53
ac	0.21	0.48	60.00
ad	1.47	4.46	<.1
ae	1.89	5.36	<.1
bc	0.55	1.29	20.40
bd	1.13	3.53	<.1
be	1.55	4.50	<.1
cd	1.68	3.45	0.1
ce	2.10	4.18	<.1
de	0.42	1.01	31.95

3.8. The six comparisons *ad, ae, bd, be, cd, ce,* given in Table 3, prove the above statement conclusively by showing very low probabilities for the hypothesis that the differences in question are only chance effects. In other words, a highly significant value of '*t*' in the present case indicates that the differences in the average milage per gallon of petrol in the pairs compared are actual features which cannot be explained as due to sampling variation.

3.9. Again, with a degree of confidence associated with eighty to eighty-five per cent, it can be said that a 'good' concrete road gives better milage than a 'fair' concrete road (comparison *ab* above) but the existence of a difference cannot on the evidence available be taken to have been conclusively established. In fact, the actual difference brought out is only 0.34 miles per gallon. Since the data available here are not homogeneous enough to give a conclusive finding on a difference in performance, which apparently is of a small order of magnitude, a controlled experiment becomes necessary to confirm these observations.

3.10. Therefore, under Indian conditions, road surfaces can be broadly classified into the three groups given in Table 4, in which the difference in performance between the first two groups is yet to be conclusively established.

TABLE 4

Milage Per Gallon of Petrol

Concrete or Black-top Good Surface	Concrete or Black-top Fair	Water-bound macadam Fair
12.42	12.05	10.92

3.11. It seems that much is not likely to be gained* by choosing between black-top and concrete surfaces under otherwise identical conditions so far as petrol consumption is concerned. Thus, though the data supplied by the operators throw some light on this subject they are extremely inadequate

*It must be specially noted that we are here only dealing with petrol consumption. The comparison between the surfaces with regard to traffic volumes, length of life, economics of maintenance etc. are separate issues.

for drawing valid conclusions with confidence. The lack of control over the experimental data is evident in that information in respect of many important combinations is completely missing from the table: there are only six separate observations (Table 1) while the number of observations required are many more if we have to study the other factors that might affect the problem such as age of vehicles, season, traffic congestion, etc. Apart from this, the data available are also defective in having been collected under heterogeneous conditions. For example, even within the fleet of a single undertaking the standard of maintenance seemed to vary widely.

For these reasons the data collected could not meet the full requirements of the Committee.

4. SUGGESTED EXPERIMENT

4.1. The Committee having been satisfied that an experimental approach was unavoidable decided that two important nationalised undertakings should be approached to carry out an experiment on their behalf. The Committee suggested the following outline for the experiment. Each undertaking should select two (or three) roads, one each of the required surfaces, of approximately the same length, of a uniform condition throughout, with similar number of bus-stoppages, with similar conditions of traffic density, traffic offering, etc. Vehicles of the same make and type, new ones to begin with, should be run simultaneously on these roads and their experience should be recorded for assessing the relative financial effect of the different surfaces.

The difficulties arising in following this outline and the considerations to be allowed to influence the design are discussed in the following paragraphs.

4.2. The modern trend is to prefer the method known as Operational Research to a purely laboratory experiment approach. The research is conducted under actual field conditions, the experiment being suitably designed to give a complete and accurate picture of the effect of the variables involved. In so far as the present experiment is an *ad hoc* scheme, it might perhaps be held that it is a laboratory experiment of the outmoded type. On the other hand, it is planned in the actual field under normal operating conditions, and to this extent it might equally well be considered a scheme of Operational Research.

5. BASIC APPROACH IN FACTORIAL DESIGN

5.1. In an experiment designed to study one or more out of a large number of varying factors it has been the practice, during the course of observations, to vary at a time one only of the factors involved and keep all the rest constant. But recent researches have shown that not only is such a restriction in procedure unnecessary, but also that it is undesirable. The modern trend in experimental design is to vary all the relevant factors simultaneously so as to secure data under as many factor-combinations as possible. In statistical language, the interaction effects would thus be brought out without loss of information regarding the main effects. The essential principle of a factorial experiment is that if there are a number of relevant factors (say m) to be studied, each of which is capable of assuming one or more levels (say n), the experiment has to be designed to study the different possible combinations ($m \times n$) so as to obtain all the comparisons required.

The above is illustrated by assuming a simplified version of the proposed investigation. The two 'factors' affecting operation costs are the type of road surface and its condition. Each of these is capable of being in one of three 'levels': namely, concrete c, black-top b, or water-bound macadam w, and the condition may be good g, 'fair' f or 'bad' b. The two factors each at three levels will give in all the following combinations, *viz.*,

$$S_{cg},\ S_{cf},\ S_{cb},\ S_{bg},\ S_{bf},\ S_{bb},\ S_{wg},\ S_{wf},\ S_{wb}.$$

where S_{cg} represents Surface S with Concrete c and in Good Condition g and so on. Note that b means 'black-top' or 'bad' according as it occurs first or second in the suffix. In case it is possible to procure data about the cost operation (e.g. fuel consumption, life of tyres, etc.) for all these nine combinations, it will be easy to estimate and compare the influences of both the factors simultaneously from the same experiment.

5.2. An important point is that the effect of each factor is available from all the nine combinations as if the whole experiment has been devoted only to study of either of them. For, if the data are arranged as shown below, comparison of the column totals will give the relative performance of the different surfaces, namely, concrete, black-top and water-bound macadam :

S_{cg}	S_{bg}	S_{wg}
S_{cf}	S_{bf}	S_{wf}
S_{cb}	S_{bb}	S_{wb}

The same data, if arranged as shown below, will bring out the effect of the other factor, viz., condition of surface.

S_{cg}	S_{cf}	S_{cb}
S_{bg}	S_{bf}	S_{bb}
S_{wg}	S_{wf}	S_{wb}

The column totals now measure the relative response of the different conditions of surface, viz., good, fair and bad.

5.3. In fact, every observation in a factorial experiment supplies information on each of the main factors which the experiment is designed to study. While the experiment thus provide a sufficient number of observations to study the main effects accurately, opportunity is taken to permute the levels among the observations in a manner designed to throw light on the effects of the levels as well as the variation between levels in regard to each main factor or between main factors in regard to each level—what is known as 'interaction.' With the large number of observations the necessity for further replications in respect of the main factors is thus reduced.

The effect of 'interaction' cannot be studied properly when only one factor is varied at a time. A concrete road may be observed in an experiment to be more economical than a water-bound macadam road but this might not be true for all conditions of the surface. For instance, a bad concrete surface might conceivably be worse than a good water-bound macadam. This aspect of differential response is likely to be missed unless the combinations of the various factors are studied. Thus a factorial arrangement not only provides greater statistical efficiency and broad-basedness but also a wider inductive basis for the conclusions. The aim in the design of a factorial experiment should be to bring into its scope all the relevant factors, and in all their relevant levels, subject, of course, to the consideration that the introduction of additional factors implies increased time which increases the cost of the experiment.

6. DETAILS OF DESIGN

6.1. It is well known that an improvement in the surface of a road leads to economy in the cost of operation in the following respects :

(1) Decrease in consumption of fuel and lubricants ;

(2) Decrease in wear and tear of tyres ;

(3) Decrease in consumption of spare parts ;

(4) Increase in vehicle speed which leads to economies through increases in vehicle utilisation, and consequent savings under:

(a) insurance,

(b) vehicle tax, wheel tax, etc.,

(c) interest on capital (on vehicles),

(d) operational staff, and

(e) maintenance and workshop staff.

There are thus four main factors and each of them has to be studied individually.

6.2. There are the following disturbing factors which have to be eliminated or whose effects have to be estimated for a proper appreciation of the main factor effects:

(i) difference in types of vehicles,

(ii) variation in ages of vehicles,

(iii) seasonal effects and weather conditions,

(iv) operating conditions including influence of driving technique and driving standards,

(v) variation in standard of maintenance,

(vi) differences (in loadings),

(vii) differences in route characteristics and traffic congestion, and

(viii) fuel and lubricating oil characteristics.

In carrying out a controlled experiment, the effects of the above disturbing factors have to be eliminated. Because of the necessity for restricting the experiment to reasonable limits, it is out of question to study interaction effects of these disturbing factors. Their effect can be eliminated by the following methods.

Where the effect is controllable it should be kept constant, e.g. (i) and (ii) can be controlled by using the same make of vehicles which are of the same age, say, new vehicles. So also (v), (vi) and (vii) should be controlled, the uncontrollable part of the variation being allowed for in the experimental design. Item (iv) can be controlled by a "combinatorial design" in respect of the assignment of drivers, etc., while item (viii) can be controlled

in the present case, purely as a matter of expediency, by using only a single make of fuel and lubricant.

6.3. The factors entering into the experiment and the associated levels are listed below:

	Item	Detail of Levels	No. of Levels
A.	Types of Surfaces	Concrete, Black-top, Water-bound macadam	3
B.	Condition of surface	Good, Fair, Bad	3
C.	Seasons	Monsoon Non-monsoon	2
D.	Age of Vehicles	New or old	2
E.	Make of Tyre	Assuming 2 makes used	2
F.	Condition of Tyres	New, Retreaded	2
	Number of combinations 144		

Thus there are 144 combinations, even after using broad groups such as two groups for age of vehicles and two groups for seasonal variation in a year. A design allowing for all these levels in all combinations would be unmanageable, time-consuming and too costly. Some simplification is obviously desirable and the first step is to combine items *E* and *F*. Together they give four levels in all and since a vehicle has four tyres (in some cases six) these four levels can be simultaneously tested on each vehicle, should this at all be necessary. It might even be that the main questions can be answered without studying these levels *e.g.* by using new tyres of a single make only. So there will be 36 levels which involve primary factors relevant to the problems to be studied and further pruning is not possible.

6.4. In this connection there are certain important points to be considered. The condition of a road surface cannot be altered haphazardly. If we start with a surface in good condition, its condition might become 'fair' after some time and 'bad' after a later period. These periods would depend upon the kind of surface, the traffic intensity, etc., and the actual durations have to be estimated. Moreover, the order in which the three levels appear is a natural one and cannot be changed except in one controllable way, namely, that if we start with a bad surface, we can ensure, with the co-operation of the road engineer, that the surface is made fair or good whenever desired. Thus improvement to a better level is possible but not the reverse. Also, a bad or fair surface can be maintained without altering its condition for as long a period as is necessary. But even if these considerations are fulfilled, the design would not be a very satisfactory one in the sense that the investigation is likely to cover a long period. We have, therefore, to rule out the possibility of confining the experiment to a particular route and vary the surface conditions from time to time. In other words, the choice of separate routes with varying surface conditions is essential.

6.5. Again, one of the chief objects of the investigation is to find out the life of tyres for various types and conditions of surfaces. For this purpose it would be necessary to run vehicles under each of the different conditions till the life of tyres is exhausted. That is to say, a tyre would have to be worn to destruction on a 'good' condition surface, another should be worn to destruction on a 'fair' condition surface, and so on. Unless we are prepared to prolong the experiment, this means that the trials should be carried out simultaneously on the different kinds of surfaces with different surface conditions.

6.6. The requirement, therefore, is that nine separate routes (3 types of surfaces and 3 conditions of surfaces), three of which will be concrete, three black-top and three water-bound macadam, and which are similar in character with respect to length, traffic pattern, load, stoppages, etc., should be selected. Each group of three will have one representing good, another fair and the third bad surface condition. All the roads should also be of similar characteristics with regard to gradient, curvature, etc., in fact, in respect of all the other controllable factors in the experiment as far as it is practicable. Vehicles allotted to particular roads should not be interchanged or substituted. The same make of fuel and lubricant should be used in all vehicles.

6.7. Two vehicles of different ages can be run on each of these routes throughout the year and monthly observations of the various factors may be made. It may be observed that this arrangement would require as many as 18 buses and 9 similar routes which may prove to be costly and unmanageable. To start with it is suggested that two conditions of surfaces *good* and *not good* (by which is meant a road somewhere between 'fair' and 'bad' conditions) may be tried instead of three. This will not only reduce the cost of the experiment but at the same time obviate the necessity of running new vehicles on very bad roads. Simplification, if necessary, can be made in respect of the age of the vehicle also. If it is difficult to set apart so many vehicles for this experiment, only new vehicles — one on each route, may be tried. In that event, the experiment would not be exhaustive but it would nevertheless cover a fairly good ground. Further enlargement of the scope of this experiment would involve too

much cost and it is desirable to dispense with absolute replication altogether. The numerous higher order interactions available, the effects of which are primarily due to error, may be used to provide a suitable estimate of error. Moreover, though replication as such, for its own sake, is not incorporated in the design, the monthly observations within monsoon or non-monsoon months (or for the whole year in case there is no significant variation between monsoon and non-monsoon periods) are virtually replications for analysing consumption of petrol and spare parts. Similarly, while comparing lives of tyres under different conditions and types of surfaces, the data for the four tyres on a vehicle (or at any rate two in case two types of tyres are used) form replications. Thus, the design even without replications will provide sufficient number of degrees of freedom for the estimation of error and the drawing of valid conclusions with confidence.

7. INSTRUCTIONS FOR EXPERIMENT

The details of the proposed experiment worked out in the form of instructions to the operating concerns are given in the Appendix. The chief purpose in presenting this Paper is to invite criticism of the design so that the design of the experiment could be finalised after giving due weight to all the criticisms and suggestions received.

In conclusion, it may be mentioned that although only two operating concerns are being invited to carry out the experiment, it will be extremely useful if other concerns who are in a position to carry out the experiment also volunteer to carry out a similar experiment. In order to maintain comparability it is essential that the experiment should be carried out on the same lines as those detailed in the Appendix (in the form in which it is finalised hereafter). In case any one would like to make proposals for carrying out the experiment on different lines, but yielding comparable data, these would also be extremely useful.

Appendix

FACTORIAL EXPERIMENT FOR ASSESSING THE COST OF OPERATION OF MOTOR VEHICLES ON DIFFERENT TYPES OF ROAD SURFACES.

1. The experiment is to be carried out for a period of two full years at least. The period may be longer, but on no account should it be less.

2. A year is to be treated, for the purposes of the experiment, as consisting of two halves, one of which is rainy and the other non-rainy or dry. In Bombay region, the rainy season starts on 15th June and ends on 15th December. In the U.P., the rainy season starts on 1st July and ends on 1st December.

3. The experiment should start in Bombay on 15th December 1955 and in U.P. on 1st December 1955 or on other convenient dates not far removed from these dates. The idea is to work in alternate periods of six months each, each of which will be a rainy season or non-rainy season as the case may be. A few days difference in choosing the date for the beginning of a season may not make much difference, but care should be taken, once the experiment has been commenced on a particular date, to take the season as lasting for consecutive exact six-monthly periods therefrom.

4. Each undertaking should choose six routes, all on plain (non-hilly) terrain. All the six routes should be of very nearly the same length and should be such that one or more buses can be run exclusively on those routes. For example routes of length 90 miles to 150 miles, may be satisfactory. An illustrative example might be the Satara-Pandharpur route in Bombay or Delhi-Haridwar route in U.P. The six routes chosen should have the following surfaces and conditions of surfaces :

Route	Surface	Condition
1	Cement Concrete	Good
2	Cement Concrete	Average
3	Black-top	Good
4	Black-top	Average
5	Water-bound Macadam	Good
6	Water-bound Macadam	Average

A good road means one which is in very good condition throughout, that is, one on which an ordinary light motor car can maintain a speed of 30 miles or more, throughout its length, except for traffic obstructions. If portions are bad on account of repairs or otherwise, such lengths should not normally exceed 10 per cent of its length.

An average condition means that the road is in a deteriorated condition and a light motor car cannot maintain, say, a speed of more than 20 miles throughout its length. In case of doubt, it is better to choose a road which is known to be definitely bad. That is, the 'good' road should err, if at all, on the side of being too excellent, while the 'average' should, if at all, err on the side of being too bad. In fact, there would be no objection if 'average' is read as 'bad', but the difficulty is that it may be difficult to find a road that is really 'bad' and is kept bad for the whole duration of the experiment.

It would be desirable to arrange with the engineering department to ensure that they do not make changes in the road surface which vitiate the experiment. Their co-operation should also be sought in maintenance of the status quo, i. e. in seeing that the 'good' road is kept 'good' and the 'average' road as 'average'.

5. On each route, as many vehicles as may be required may be put, but for the purposes of the experiment, at least one per route, if not more, should be earmarked. All the vehicles should be of the same make and type and year model. If only one vehicle is put on a particular road, it must, to begin with be 'new'; a vehicle being defined as 'new' if it has not run more than 20,000 miles in all, and if it has not undergone any major item of repair. If more than one vehicle is put, the other must be an 'old' one, this being defined as one which has run more than 60,000 miles and less than 100,000 miles. Again, the vehicle should not have undergone any major repair due to having met with an accident, but, of course, it can be one which has been thoroughly overhauled and engine reconditioned.

6. When starting the experiment, all the vehicles concerned must be fitted with new (not retreaded) tyres, all of the same size and make. There is, however, no objection to two makes being experimented upon, in which case half of the tyres must be one make and half the other. In the latter case, fit the same make of tyre on each lateral side of the vehicle (i.e., one in front and one or two as the case may be, in the rear on each side). Changing over of the tyres should be done systematically, at almost the same time on all the vehicles, and the same tyres should be kept on the same vehicles in all cases.

7. The spare tyres do not come into the experiment and they may be of any make ; but, the spare tyres should not be changed over with the experimental tyres. If, at any time, due to puncture or otherwise, the spare tyre has to be fitted, see that the experimental tyre is replaced as soon as practicable and keep a separate note of the milage run in the meanwhile, so that the necessary corrections may be allowed for in the calculations.

8. See that the milometer is working alright and that the milage readings are regularly taken.

9. The tyres should always be kept properly inflated and at the same pressure on all tyres of all the vehicles. If a difference in pressure between the front and the rear wheels is required, see that this is maintained in all cases always.

10. Any defect becoming apparent in a vehicle which might affect tyre life (e.g., wheel alignment), should be attended to as expeditiously as possible.

11. The tyres having been fitted in new condition at the commencement of the experiment, the milages for which each tyre runs before it becomes unserviceable, should be noted. A tyre becomes unserviceable if it is worn out, or bursts or receives a cut which cannot be repaired satisfactorily. A tyre should be treated as worn out, when the treads on it cease to be visible, even though it may be possible to carry on with that tyre for some time more.

12. It will be permissible in unavoidable cases, to replace a particular vehicle with another of the same make, age-category and condition. But this should normally be avoided and the same vehicle should be made to run on the particular route during the whole duration of the experiment. Where a new vehicle of the same make is substituted, see that there is no difference in the specifications of the engine, carburettor etc. and also fit the tyres of the old vehicle to the new one so that the experiment in regard to the tyre is continued. (This is not necessary if a vehicle is only temporarily out of service for repairs etc. In this case, any substitute vehicle temporarily put on the road, would not be taken into account and the original vehicle would be deemed to be still in service, but running only zero miles during the temporary period).

13. The same make of fuel and lubricants should be used throughout in all the vehicles. The grade of oil and lubricants may, however, be changed seasonally but the change-over should be made at almost the same time. Abnormal engine oil leakage should be attended to expeditiously.

14. The engine of every vehicle should be kept in good tune always and periodical check-ups for this purpose are desirable.

15. The spare parts issue and maintenance programme should be such that the vehicle is maintained in a normally satisfactory condition. All normal repairs necessary should be carried out in proper time.

16. As far as possible, allot good experienced drivers for the experiment; a good experiment can easily be spoilt by bad execution. The drivers should also be asked to follow correct driving techniques.

17. The drivers should be asked to maintain safe maximum speeds. That is to say, while dangerous speeding is to be avoided, they should run in a normal way and not act as road-hogs.

18. In order to eliminate driver effect, the drivers should be systematically rotated with respect to the vehicles and also if possible, with respect to routes.

19. It might be advisable to entrust a responsible officer with the implementation of these requirements and for continuous control and direction of the experiment.

20. The following observations should be taken :

(1) For each trip.

(a) Milometer reading as at the commencement.

(b) Exact time of commencement of the trip.

(c) Exact time of intermediate halts.

(d) Exact time of end of trip.

(e) Milometer reading as at the end of the trip.

(Alternatively, the exact time of starting and stopping at each halt on the way may be noted).

The object is to work out the exact speed attained in running on the roads. The daily milometer readings will be a useful check. The times of running should exclude stoppages times. The persons concerned should be asked to record the times of arrival and departure accurately and on the same watches/clocks.

(2) For every instalment of fuel (petrol) delivered to the vehicle, the date on which delivered, the quantity delivered (in gallons), price, the milometer reading at the time of delivery. If petrol is taken loose in cans, the same particulars should be taken but care must be taken to see that the petrol taken is used wholly for running the vehicle. If petrol is required separately for cleaning etc. such petrol should be taken separately and accounted under spare parts.

(3) A separate account should be kept of all lubricants supplied to the vehicle, viz., engine oil, grease etc., mentioning the date, quantity supplied and price. Engine oil for topping up purposes should be shown separately from engine oil used for oil-changing.

(4) A record should be kept in respect of each vehicle listing the spare parts issued (or fitted), the date and the price. Similarly, all other expenses incurred on the running of the vehicle will also be shown in a list datewise.

(5) For each tyre, the size, make, date on which newly issued, price, milage run before being discarded

Paper No. 182

"COMPRESSIVE STRENGTH AS A DESIGN CRITERION FOR BITUMINOUS MIXTURES"

By

C. G. SWAMINATHAN

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SYNOPSIS

The drawbacks of an unconfined compression test as is used for determination of bituminous mixture compositions are discussed. It is suggested that, in spite of these drawbacks from a fundamental point of view, it could be used as an empirical indicator test for mixtures such as sheet asphalt or sand carpets.

1. INTRODUCTION

1.1. Amongst the various types of stability tests that have been employed for the evaluation of an asphalt mixture for road paving purposes, the simplest and the most common one is the unconfined compressive strength test. But of late, with the advent of triaxial apparatus as modified for asphalt mixtures, the purely unconfined compressive test result is not so much in vogue as one of the test criteria for the design of bituminous mixture compositions.

1.2. The object of this Paper is to find out and to enumerate the various drawbacks of the compression test as applied to bitumen mixtures and to find out, if possible, whether the compressive strength values could be interpreted in a different way and to see whether diagramatic analysis of the data could be used for specific design purposes.

2. THE UNCONFINED COMPRESSION TEST

2.1. The procedure for conducting compressive strength tests on asphalt paving mixtures differs in no way from the normal methods that are in vogue for concrete and allied materials. Cylindrical specimens 3 in. high and 3 in. in diameter as well as 2 in. high and 2 in. in diameter were used by the joint committee set up by the British Ministry of Transport and the British Standards Institution [1]. In Germany and the U. S. A., cubes of various dimensions have been tried. Because of the plastic nature of the test specimen, due importance is always given to the rate of deformation and the temperature of testing.

2.2. By doing a series of experiments on various compositions, Vokac[2] has shown that the load deformation curves in a compressive test reveal a straight line relationship initially and then a deviation from the straight line showing a decrease in increments of load, as deformation continues, until a maximum load is reached. The load then starts to decrease at an increasing rate as deformation continues entering another region where load is proportional to deformation, after which the increase in load diminishes causing the curve to flatten out. The modulus of deformation which is derived from the slope of the load-deflection curves beyond the deformation at which compressive strength is measured is claimed as an evaluation of the mixture after the packing arrangement of the aggregates is disturbed. Vokac goes on further and gives a set of equations for obtaining the various properties of each mixture, and he has deduced that the resistance offered by a bituminous surface to traffic can be measured in a resultant compressive stress regardless of the nature of the ingredients used or any other individual characteristics.

It has been found by other investigators that the reproducibility of the results in a pure unconfined compressive test for bituminous mixtures is very poor and that the method of compaction, density and the size of the sample etc. influence the test values a great deal and that although the results can be expressed on a lb per sq. in. basis, they are not fundamental or rational in nature.

3. UNCONFINED COMPRESSION TEST IN RELATION TO OTHER TYPES OF STABILITY TESTS

3.1. The criticisms mentioned above could be applied to the various other forms of stability test such as the Extrusion and Indentation tests or the Flow tests. Almost all these test values are influenced considerably by the methods of compaction employed, the size of the samples, the rate of deformation and the temperature of testing, apart from the composition of the mixture. The only justification for these forms of stability tests seems to be that the factor of plasticity or viscous resistance which is one of the salient characteristics of a bituminous mixture is brought out more clearly in a flow type of test rather than in an unconfined compression test, which by giving the test data in the form of an ultimate load at fracture, stresses more on the stability of the mineral aggregates rather than on the cohesive properties of the binder.

3.2. It is obvious that the unconfined compression test is only a particular case of the triaxial test, with zero lateral pressure. Further, the test results of a triaxial shear are usually expressed in terms of the two physical characteristics viz. cohesion 'C' and the angle of internal friction 'ϕ'. Because of this the advocates of the triaxial test claim a more rational approach to the problem of the design of an asphalt mixture than those of the unconfined compression test. The various forms of the triaxial tests in use yield the same primary properties viz., 'C' and 'ϕ' and the same are utilised as design criteria by using different equations. Smith[3] makes use of equations developed from the mathematical theory of elasticity. Nijboer[4] uses Prandtl's equations for the same purpose and McLeod[5] bases his findings on the geometric relationships of the various quantities in the Mohr's diagram. And although the equations that have been developed by the above mentioned workers are quite rigorous in character, they are not above criticisms.

3.3. The theory of elasticity frequently does not hold good in analysing a flexible pavement structure, particularly with regard to deflection and uncertainties may result from the application of this theory to stressed materials close to the loaded area; because viscous resistance is an important factor in bituminous pavement design, Nijboer has quite successfully attempted to bring this factor in his equations. But there are experimental evidences to show that his findings may not hold good universally. Coulomb's Law which is used for the envelope of the Mohr's circles, has very often been questioned as regards its validity. A curved Mohr envelope has been the finding of many an experimentor on asphalt mixtures. The use of stress analysis for strength computations has also been questioned. Further, Housel[6] has strongly challenged the application of Mohr's circles for granular mixtures.

4. SHORTCOMINGS OF THE COMPRESSION TEST

4.1. However, based on Mohr's diagram the inadequacies of an unconfined compression test as a stability criterion for the design of asphalt mixtures have been proved[7]. Geometrically, it is possible to have three distinct compositions of an asphalt mixture (with varying proportions of the ingredients) having the same unconfined compressive strength but different values of cohesion and internal friction. Similarly there can be three dissimilar and totally different (based on the values of C & ϕ) bituminous mixtures possessing three distinctive unconfined compressive strengths but showing the same principal stress difference for a given lateral pressure in a triaxial compression test. So it follows that wide variations in the compositions of an asphalt paving mixture could be made with little or no change or effect whatsoever on the value of the unconfined compression test.

4.2. It is apparent that the above criticism is based on the assumption that the stability of a pavement is solely governed by factors such as cohesion and the angle of friction. This assumption implies that weightage should be given to and emphasis laid on the so called fundamental concepts of the material and not on a strength factor such as the unconfined compression value. But, when it is realised that the definitions of cohesion and angle of internal friction as based on Coulomb's Law are not so very rigorous and definite as one would like them to be, the high amount of relative weightage that is attached to these primary physical properties of the material as against its compressive strength, seems to be not wholly justified.

4.3. However, if it is still assumed that the values of cohesion and friction are definite constants for a given mix composition, are not affected by considerations of anisotropy inside the mass and are independent of the size of the sample, it may be possible to work out the values of C and ϕ based on compressive strengths. If two samples of the same mixture are compacted exactly under identical conditions to the same density and then tested at the same temperature and by employing the same rate of deformation—only difference being in the size of the specimen—and if the end frictions between the faces of the specimen and the platens of the testing machine are eliminated completely by

providing rubber sheets or some such adequate device, the values of cohesion and friction may be worked out as shown in Para. 5.2.

5. DETERMINATION OF C and ϕ FOR DESIGN OF MIXTURES

5.1. If a cylindrical sample of diameter 'd' and height 'h' with a low ratio of height to diameter is taken, it is obvious that due to the interference of the platens, the plane of failure on application of the ultimate compressive strength intensity of 'p' will be forced along a slope making an angle = '$\propto$' such that $\tan \propto = \frac{h}{d}$, instead of an angle $= 45° + \frac{\phi}{2}$ where ϕ is the angle of internal friction. And if τ and σ are the shear strength intensity and the normal stress intensity for the plane of failure, it follows that:

$$\sigma = \frac{p}{2} (1 + \cos 2\propto) \qquad \text{......(1)}$$

and,

$$\tau = \frac{p}{2} \sin 2\propto \qquad \text{......(2)}$$

At the time of failure, assuming Coulomb's Law, it is known that:

$$\tau = C + \sigma \tan \phi.$$

where C and ϕ are the values of cohesion and internal friction for the mixture in question. From the above three equations it follows that:

$$\frac{p}{2} \sin 2\propto = C + \frac{p}{2} (1 + \cos 2\propto) \tan \phi \qquad \text{......(3)}$$

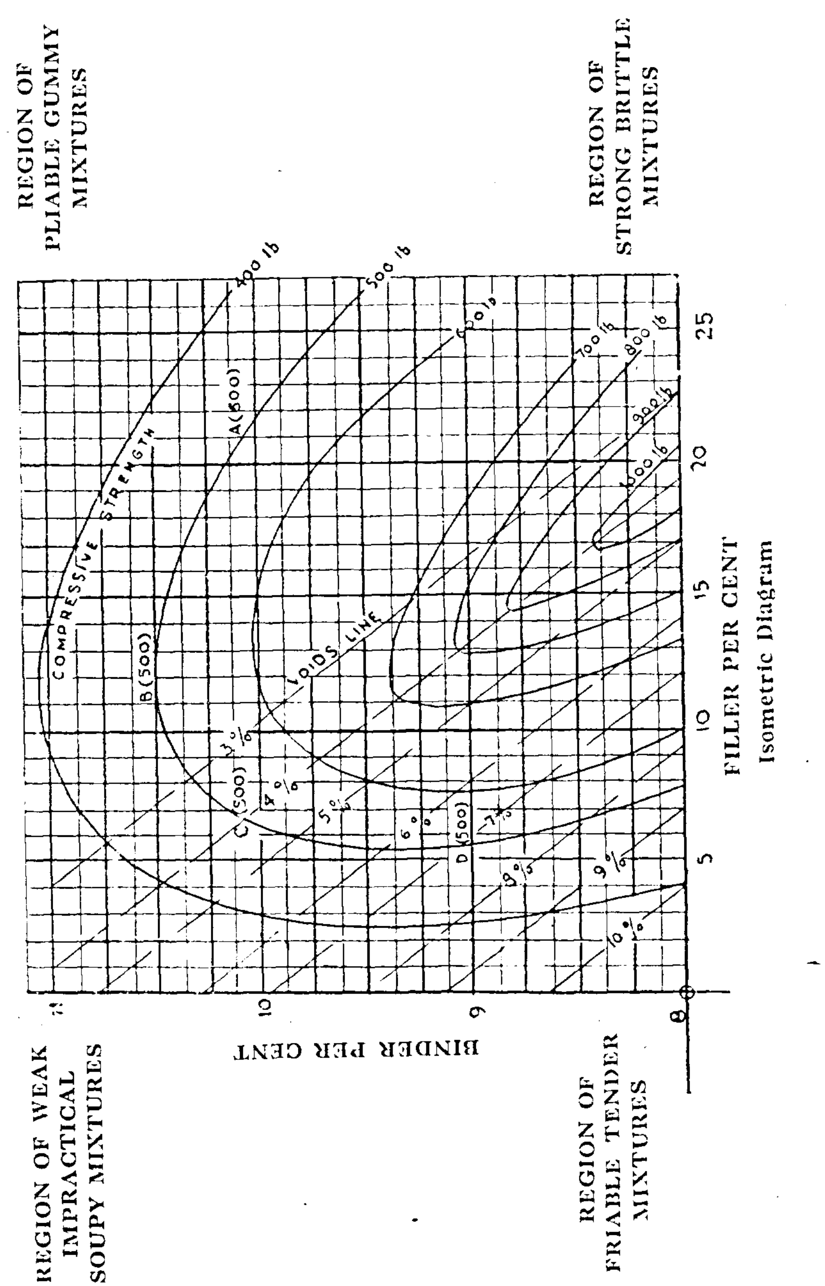
5.2. If two samples of the same mixture, but of sizes h_1, d_1 and h_2, d_2 are taken and if the compressive strength values are obtained as p_1 and p_2 respectively, the values of C and ϕ can be obtained readily by substitution in Equation (3) and by solving the simultaneous equations. These values of C and ϕ could be used for the mix design by using charts similar to those of McLeod or of the Asphalt Institute. But then, it may be criticised that these values of C and ϕ are not truly correct because of the large number of assumptions involved which are too audacious in character.

5.3. Perhaps, because the nature of an unconfined compression test is such that it is intended only for the determination of a strength characteristic and because an asphalt paving mixture is a plastic mass, it may not be possible—nor can it be justified—to evaluate correctly the values of the so called physical properties of the material based on a compression test data. But when a load deformation curve for a compression test on a sample of asphalt mixture is drawn, a characteristic shape is obtained, showing an initial rise and a subsequent fall. This indicates the probable existence of either a definite stress strain relationship for the mixture or a more intrinsic property for the material. And it is because of this that earlier investigators were prone to treat a bituminous mixture on a par with other engineering materials. And again inspite of the advent of other forms of stability tests, there is still a tendency to give weightage to the compressive strength value. Possibly a detailed and accurate study of the total work done before fracture sets in or between any load increment intervals, might give a different interpretation or understanding of the phenomenon of stress strain relationship within the mass.

6. USE OF ISOMETRIC DIAGRAMS BASED ON COMPRESSIVE STRENGTHS FOR DESIGN OF MIXTURES

6.1. Till such a time when compressive strength values could be used to give an insight into the fundamental properties of the test specimen, only the ultimate load at failure will be recorded, and with these values Vokac[8] has developed a graphical method by which a mix composition could be worked out. The principle of the method is similar to that of drafting of isobars or isotherms in weather maps or contours on a topographical map.

6.2. In the diagram on page 147 the vertical axis represents the binder percentage and the horizontal axis the filler percentage for a sheet asphalt mixture. The sum of these two subtracted from 100 gives the percentage of sand. Taking a set of different compositions of filler and bitumen, their compressive strengths are recorded and the points plotted such as A, B, C, D, etc., depending on their percentages of bitumen and filler which are their coordinates. Their corresponding compressive strengths are written in brackets. When a sufficient number of points is obtained, lines connecting all points having the same compressive strengths are drawn as shown in the diagram. These lines are the iso-lines. At the four corners of the diagram are written the salient characteristics of a representative mixture of that region. So the composition of any mixture required for any particular specification as given by each of the four zones, could be obtained based on a compressive strength of three or four



trial mixtures. To make this method more useful, lines joining all points having the same voids percentages are drawn (inclined lines of the diagram) and with the help of these lines, the area for the choice of a mix composition could be narrowed down still further and undesirable mixtures could be eliminated. This method could be made versatile by drawing the iso-cost lines as well. A similar chart could be drawn for selection of quantities of fine aggregates, coarse aggregates etc. required for an asphaltic concrete though it may be asked as to whether crushing of aggregates would not be the result under direct compressive loads. For sheet asphalt or sand carpets, compressive strengths may be relied on and Rice and Goetz[9] have used a similar method for their investigations for lake and waste sands.

7. CONCLUSION

However, as it stands at present, the compression test for asphalt mixtures has been shelved and is not very much used as a stability test for evaluation and for purposes of bituminous pavement design. But the fact remains that it is very simple to carry out either in the laboratory or in the field and that it is applicable to a very wide range of bituminous materials. It is true that it is not capable of giving an insight regarding the fundamental properties of a mixture with the result a definite design based on compressive strength, alone will not be a rational or scientific one. It is equally true that it does not at all represent road conditions where the surfacing material is supported laterally, whereas in the test the sample is unconfined. Yet, when a series of tests solely for purposes of determination of the composition of a mixture such as one required for bituminous stabilisation of a sandy soil has to be conducted, an indicator type of test like the unconfined compression may be thought of—in spite of its drawbacks—and used to the best advantage in conjunction with a set of curves such as the isometric lines. The rational design however will have to be based on a different finer test data.

ACKNOWLEDGMENT

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Paper No. 183

"PRESTRESSED CONCRETE BRIDGE ACROSS THE MAHI RIVER AT VASAD ON NATIONAL HIGHWAY DELHI—AHMEDHABAD—BOMBAY"

By

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AND

I. K. NAIK

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SYNOPSIS

The Paper describes the design and construction of the prestressed concrete bridge across Mahi River at Vasad on the Delhi—Ahmedabad—Bombay Road, twelve miles to the north of Baroda. The Bridge consists of 16 spans of 110 ft centre to centre of piers each and is founded on twin 8—shaped wells. The superstructure of the bridge is 9 in. thick R.C.C. slab cast in situ over two precast prestressed (Lee McCall system) I-shaped girders.

An important feature of the bridge is its great height (66 ft) above the river bed level and consequent difficulties in construction. The girders were cast and prestressed on the bank and then launched into position by an ingenious method by successfully moving a launching girder from span to span and utilizing the previously launched girders to carry the track over which subsequent girders were conveyed by trollies.

Full details of precasting, prestressing and launching of girders, weighing 95 tons each, from the place of casting to their final position in the bridge are given.

1. MAHI RIVER AND LOCATION OF BRIDGE

1.1. The river Mahi forms a natural boundary between Kaira and Baroda districts of Bombay State, and is also the boundary between North and South Gujerat. The river is popularly known as Mahisagar because of its great width in this area. It flows from the Malwa plateau of Central India through rocky country up to Sewalia in Gujerat and thereafter through loamy country until it joins the Gulf of Cambay, Plate I. The National Highway, Delhi—Ahmedabad—Bombay, passes through Baroda and crosses the river at Vasad. The crossing is unfordable during six months of the year, therefore an improvised temporary crossing is provided during these six months of the year to cater to the needs of road traffic. Construction of the bridge at this crossing was given the topmost priority as it linked two highly developed regions on the National Highway route.

2. HYDRAULICS OF THE RIVER

The river has a catchment area of 12,000 square miles at Vasad and a high flood discharge of 12,00,000 cusecs. The width of the river is 1,760 ft from bank to bank and the high flood depth is 64 feet.

3. TYPE OF BRIDGE

3.1. The railway bridge downstream has eight spans of 220 ft centre to centre of piers. Sixteen spans of 110 ft each centre to centre were selected for the road bridge so as to keep the alternate piers of the two structures opposite each other. Tenders for the superstructure were invited for the following alternatives :

(a) Steel plate girder—balanced cantilever design—
Four girder system.

(b) R. C. C. balanced cantilever—three girder system.

It was ultimately found advisable to adopt the prestressed concrete girder design (Lee McCall system), as it solved successfully the problem of arranging centerings for a span of 110 ft at a height of 69 ft and thus led to a greater elegance in construction

and considerable saving of time. The lowest tender for the departmental design was Rs 10,73,815 for the R. C. C. balanced cantilever, whereas the tender for the prestressed girder was for Rs 11,13,912, i.e., Rs 40,000 more, an amount quite insignificant considering the magnitude of the work and the advantages enumerated above. A comparison of costs of the superstructure (16 spans) for these two types of construction on the basis of fair rates is at Appendix I.

4. GENERAL DESCRIPTION OF THE BRIDGE

4.1. The bridge is 1,765 ft between the outer faces of the two end piers and consists of 16 spans 110 ft centre to centre of piers, Plate II. The piers are of mass concrete, 1 : 3 : 6, 10 ft 6 in. by 27 ft 6 in. at the base and 5 ft by 22 ft at top. The batter is 1 in 24. The piers have a lightly reinforced cap 3 ft in thickness. The foundation of each pier consists of twin 8-shaped wells with dredge holes 14 ft diameter and a steining thickness of 2 ft with overall dimensions of 35 ft by 18 ft with a waist of 13 feet. The wells are provided with cutting edge of angles and plates, Plate II. The depth of wells is from 30 ft to 34 ft with about 5 ft in stiff clay, Plate III. The bottom plugs are 6 ft thick in 1 : 2 : 4 concrete. The top plugs are 2 ft thick in 1 : 2 : 4 concrete. The rest of the well space has been filled with sand. The caps are 3 ft thick of 1 : 2 : 4 R. C. C. (High Grade).

4.2. Superstructure

The superstructure comprises two I-shaped precast prestressed girders connected by cast-in-situ R. C. C. diaphragms at 15 ft intervals and decked by a cast-in-situ R. C. C. slab to give a 24-ft wide roadway, Plate IV.

All the girders are 9 ft deep with 4 ft 6 in. wide top flanges. The web thickness is 8 inches. The top flange is 4 in. thick at the edges and increases linearly to 8 in. where it meets the web. The junction of flange and the web is chamfered in order to have a smooth transition. The girders have solid rectangular end blocks for a distance of 4 ft 8 in. at each end. Each girder is post-tensioned with fifteen $1\frac{1}{8}$ in. diameter Macalloy bars. The bars, except two in the top flange, are stressed before launching. Adequate mild steel reinforcement is provided in the web and flange to take care of any adverse stresses due to handling, temperature and shrinkage. The girders have stiffeners 8 in. thick, 15 ft apart. The two girders of one span are connected through stiffener diaphragms 6 in. thick cast after they were placed in position. The girders are provided with roller and rocker bearings 107 ft centre to centre, Plate V.

5. DESIGN

5.1. The girders were designed to carry their own weight together with the dead load of the diaphragm and the slab, and also to carry the surfacing and live load jointly with the deck slab as a composite unit. The girder section could have been reduced if higher stresses had been allowed but for the sake of safety the maximum compressive stress in concrete was limited to 1,500 lb per sq. in. in the design.

To obtain the necessary bond between the girder and deck slab and to ensure that they functioned properly as a composite unit, castellations 9 in. wide and 2 in. high have been provided along the top flange of the girder at 1 ft 9 in. centre to centre. In addition to this, three $\frac{3}{8}$ in. diameter U-bars project out of the top flange of the girder into the deck slab in between the castellations. In the design of the composite unit due consideration has been given to the difference between the permissible compressive stresses in the slab and the girder, the maximum compressive stress in the slab being 950 lb per sq. in. as against 1,500 lb per sq. in. in the girder. The girders have been designed to have no tensile stress in the concrete under all conditions of loading.

The Macalloy bars employed for prestressing have an ultimate strength of approximately 66 tons per sq. in. and a 0.2 per cent proof stress of 58 tons per sq. in. and are initially stressed to 42 tons per sq. in. An allowance is made in the design for loss of prestress due to shrinkage and creep of the concrete to the extent of 15 per cent.

The design calculations of the girder are given in Appendix 2.

6. CONSTRUCTION OF FOUNDATIONS AND SUBSTRUCTURE

6.1. Well Kerb

The cutting edge of the wells was built up of angles 4 in. by 4 in. by 5/8 in. and 12 in. by $\frac{1}{2}$ in. plates. It was assembled in pieces at site and welded. The river bed was dry for eight months during construction and hence no difficulties were encountered in placing the cuttting edge in position. Bent up flats 3 in. by 5/8 in. were welded to the cutting edge to form the shoe of the kerb. One cutting edge with flats without bond rods weighed 2.7 tons. The shuttering for the kerb was of wood. Bond rods were screwed to the angles of the cutting edge. The cutting edge and kerb were kept in perfect level and concreted in 1 : 2 : 4 mix, Plate II.

6.2. Steining

The foundation well was of the shape of numeral 8, i.e. twin wells connected by a common side. Overall dimensions of a well were 35 ft by 18 ft with 14 ft diameter dredge hole and 2 ft thick steining and 3 ft thick connection wall. Reinforcement of the steining consisted of $\frac{1}{2}$ in. vertical rods at 9 in. centres and $\frac{3}{8}$ in. hoop rings at 12 in. centres, which came to about half per cent. The concrete used was 1 : 2 : 4 mix with $1\frac{1}{2}$ in. gauge Mahi river gravel as coarse aggregate. The steining was raised in stages of 4 ft using steel shutterings. To facilitate the fixing of the shuttering for the next stage, holes were left 6 in. below the top of each stage so that the shuttering for the next stage could be conveniently fixed on the concrete surface of the bottom lift and made flush with it. Timber and angle distant pieces were used to maintain the thickness of the steining and to preserve the shape of the twin well.

6.3. Sinking

The wells were to be taken to a minimum depth of 30 ft below river bed and 5 ft into stiff clay. The wells were sunk through sand, shingle, sandy clay, and stiff clay, Plate III.

Initial sinking was done by dredging up to about 20 feet. Two steam cranes with grabs of 5 cu. ft. capacity were used for the purpose, Photo 1. There was no special difficulty in sinking except in sandy clay and stiff clay. In the latter case, all operations of sinking like chiselling, sending divers with diving outfits for scooping out and cleaning lumps from beneath the cutting edge using pneumatic pavement breakers and light blasting by gelignite for loosening the clay at the bottom, were resorted to. Generally, it took about 17 days to concrete and sink a well from R. L. 23.00 to -3.50 piercing the sandy strata. No kentledge or special effort was required. The whole sinking was carried out by open dredging by means of crane and grab. It took about 15 days for the well to pierce the sandy clay strata from R. L. -3.50 to -7.91, with a kentledge of 120 tons.

The stiff clay strata was very hard and it took about 20 days to sink from R. L. -7.91 to -13.12. A total kentledge of 600 tons was put on the well, Photo 2, and 19 shots of gelignite, each of 1/16 lb were fired to loosen the clay.

The entire operation of concreting the steining and of sinking the well to a depth of 36 ft took 52 days.

6.4. Bottom Plug

After the well was sunk to the required level, bottom 6 ft of the well and the scooped out portion below the cutting edge were plugged with cement concrete 1 : 2 : 4 by using a skip box. A skip box measuring 2 ft by 2 ft by 2 ft was used and was filled with 1 : 2 : 4 cement concrete just wet mix and lowered into the well through the water by means of a crane. The box, when it touched the bottom, was made to open out automatically and the concrete was deposited in place. Divers were sent at intervals to level up the concrete. Depth was ascertained by means of soundings. After the concrete of the bottom plug had set, water was pumped out and the watertightness of the plug was tested and was found satisfactory. The strength of the concrete was tested by means of a hammer.

6.5. Sand Plug

Sand plugging was done under water as sand becomes more compact in that condition. The top 2 ft of the well was plugged with cement concrete 1 : 2 : 4.

6.6. Well Cap

The well cap is 35 ft by 18 ft by 3 ft. At the waist, the width is 13 feet. The concrete mix was 1 : 2 : 4 high grade, the maximum size of aggregate being $\frac{3}{4}$ inch. Steel shuttering was used. The reinforcement worked out to about 2 per cent. The tendered cost of one well including well cap was Rs 50,000. The average strength of steining concrete at 28 days came to 3,500 lb per sq. inch and that for the well cap concrete came to 4,500 lb per sq. inch.

6.7. Piers

The pier was concreted using 1 : 3 : 6 mix. It was raised in 5-ft stages. Shuttering of straight portion was made of steel plates while that for the curved ends was of timber with necessary angles. Concreting up to three lifts was done by manual labour on scaffolding. For heights from 15 to 30 ft,

steam cranes and bucket were used, Photo 3. At an elevation of more than 30 ft from the bed and up to the top, i.e., 66 ft, the concreting was done by a tower and a steam winch. Between the tower and the pier a chute was arranged at such an angle that no segregation of concrete could take place. The bucket was lifted through the guides in the steel tower actuated by means of a steam winch and pulley. The concrete was vibrated by means of internal vibrators.

Nominal temperature steel consisting of $\frac{1}{4}$ in. bars at 12 in. centre to centre both horizontally and vertically with $1\frac{1}{2}$ in. cover was provided on the exterior of the piers. The piers were provided with R. C. C. pier caps 3 ft thick and projecting 6 in. all round. Pier caps were reinforced both ways at top and bottom. The percentage of steel came to $1\frac{1}{2}$ per cent.

The quantities of materials used in a pier are given below :

	Per Pier
1 : 3 : 6 concrete	12,000 cu. ft.
1 : 2 : 4 concrete in pier cap	400 cu. ft.
Cement for pier and cap	75 tons
Total steel for pier with cap	3 tons
Cost of pier with cap	Rs 28,000

7. PRECASTING AND PRESTRESSING OF GIRDERS

7.1. Casting Yard

In all 32 girders for 16 spans were to be cast. It was, therefore, necessary to plan the casting in two instalments. The casting yard was established on the south bank of the river and measured 300 ft by 150 ft. A trolley line was laid centrally from one end of the casting yard to the other. The casting bed consisted of sleepers put at intervals below the girder bottom flange and along the length of the girder to distribute the girder weight evenly on the soil while concreting. At the ends timber grillages for support were provided to distribute the girder load at $\frac{3}{4}$ ton per sq. ft. The timber grillages were subsequently replaced by concrete rafts as they were found to be yielding during the curing of the girders by reason of the curing water finding access to the soil bed and reducing its bearing capacity.

7.2. Girder Shuttering

The shuttering was built up of steel plates. First the bottom flange and one side of shuttering was erected and then the Macalloy bars, inserted into sheaths of lead coated steel unitubes, were put in position with necessary spacer blocks, Photos 5 and 6.

These spacer blocks were prepared in high grade concrete and thoroughly vibrated as they were to go into the body of the concrete to form an integral part of the girder. It was quite necessary that the high tensile bars should not get disturbed due to the pouring of concrete and vibration. The parabolic shape and the exact position of the bars had to be maintained correct at every section to ensure development of the correct prestress without undue friction. The bottom side of the flange was given a camber of one in. in the centre.

The detailed arrangement of secondary reinforcement and bar ends for the end blocks are shown in Photo 4. The ends of the Macalloy bars at the bottom of the girder come out straight while others come out at an angle. In order to make it possible for the Lee McCall jack to fit to the concrete surface and pull the Macalloy bars axially, the end block had to be given a suitable shape, Photo 11.

Necessary steel anchor plates were then threaded on to the Macalloy bars and kept in position along with the shuttering. This was done in such a way that the correct length of threaded portion of the Macalloy bar end projected from the anchor plate and the anchor plate was at right angles to the projecting end of the bar. Spirals of 3/16 in. rods at the ends were provided for the purpose of uniform distribution of the pressure through the end block to the body of the girder.

Macalloy bars are supplied cut to predetermined lengths and threaded for direct use and are available in lengths up to 65 feet. Longer lengths were obtained by using Lee McCall high efficiency couplers, Plate VI.

7.3. Concreting of Girder

The girder was cast at one stretch in nine hours, usually from 5-30 a.m. to 6-0 p.m. The concreting equipment consisted of the following :

(1) two 10/7 cu. ft. concrete mixers,

(2) one double unit weigh batcher for weighing concrete ingredients,

(3) weighing arrangement for cement,

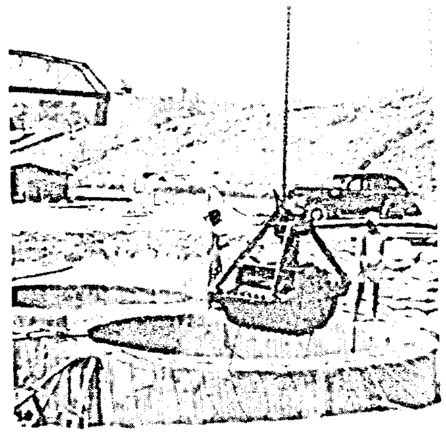


Photo 1. Open dredging in well

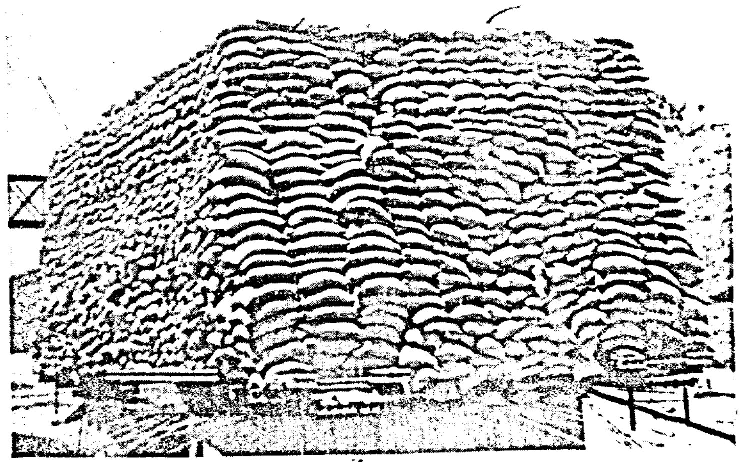


Photo 2. Kentledge for sinking well in stiff clay

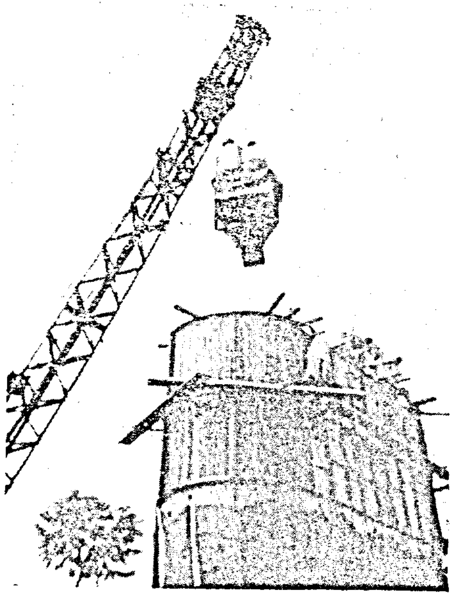


Photo 3. Pier concreting inspection at 30 feet elevation

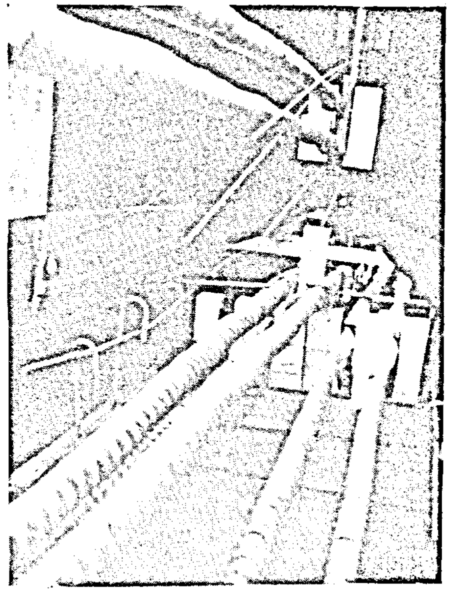


Photo 5. Macalloy bars with spacer blocks in position

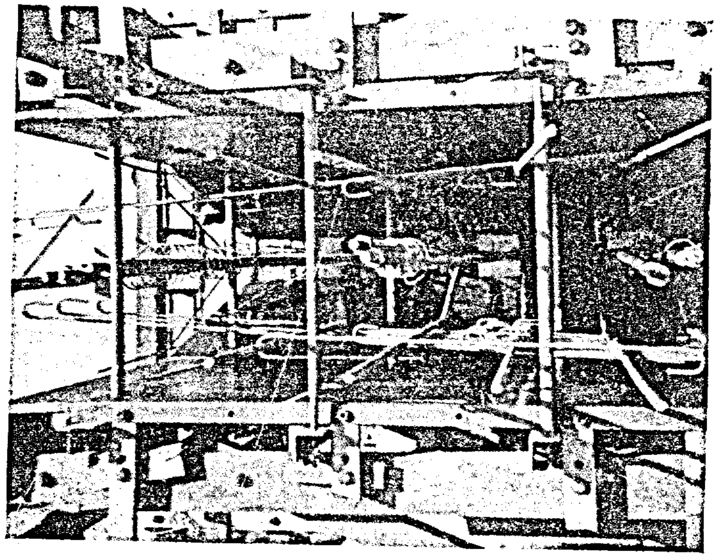


Photo 4. Girder end details

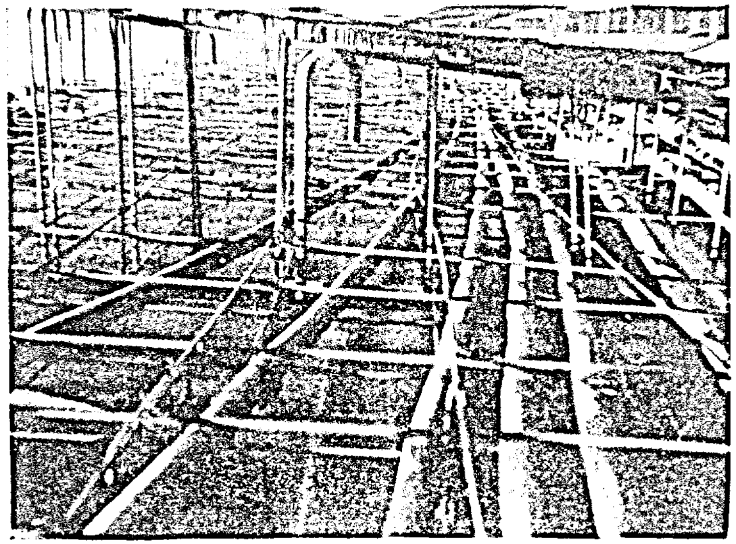


Photo 6. Macalloy bars in position

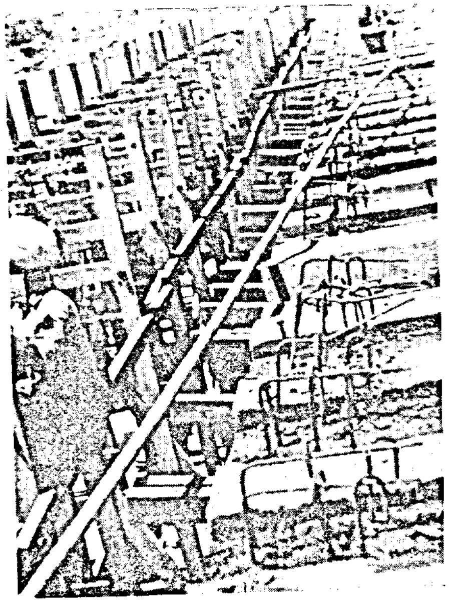


Photo 7. **Girder castellations and dowel bars**

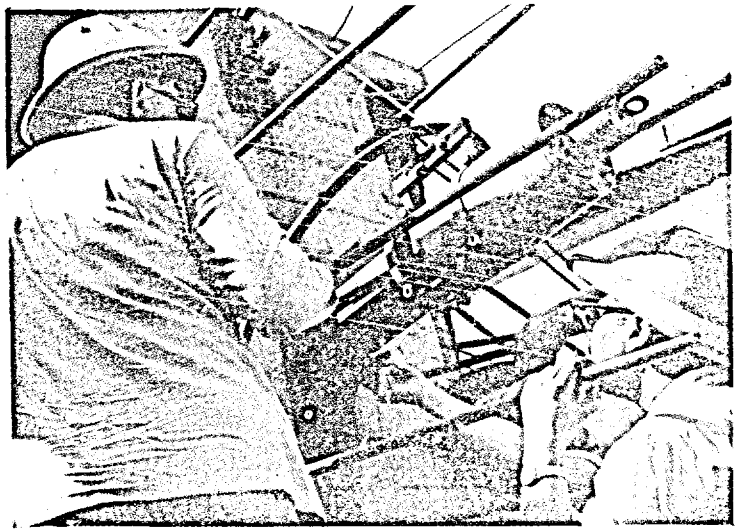


Photo 9. **Prestressing operation—reading the vernier**

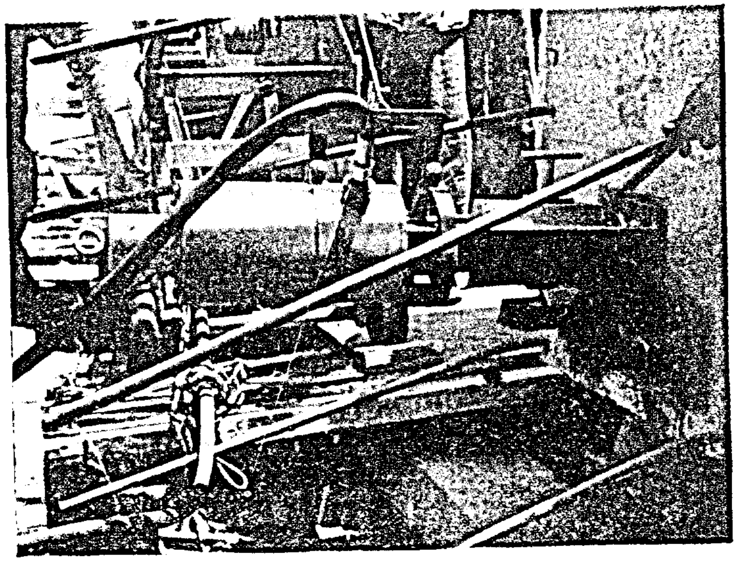


Photo 8. **Prestressing jack**

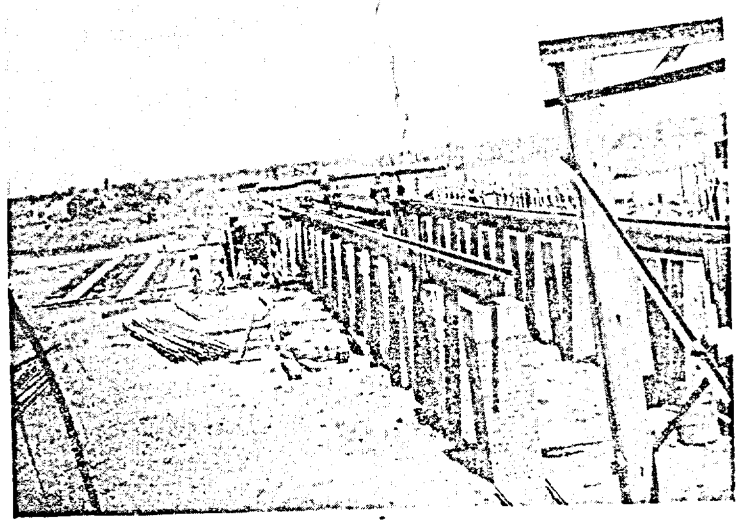


Photo 10. **Trolley track for launching**

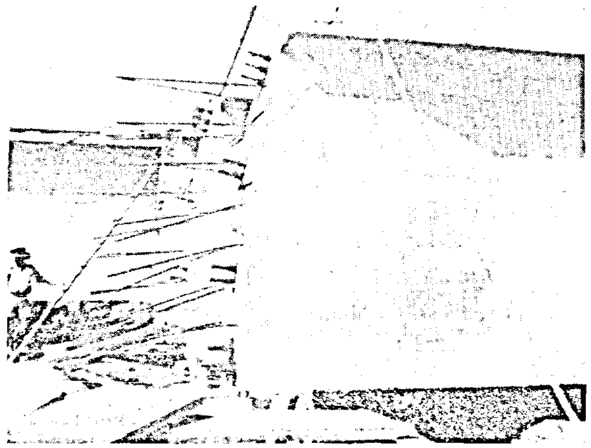


Photo 11. **Girders showing curved ends**

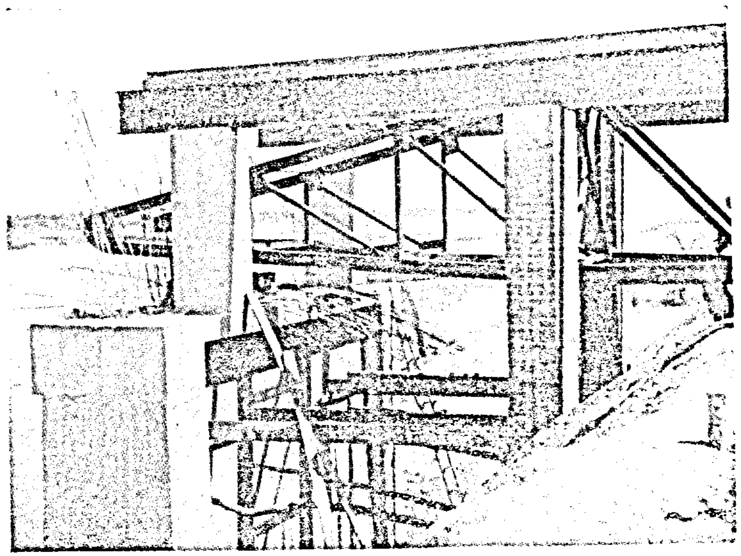


Photo 13. **Steel launching girder moving to position**

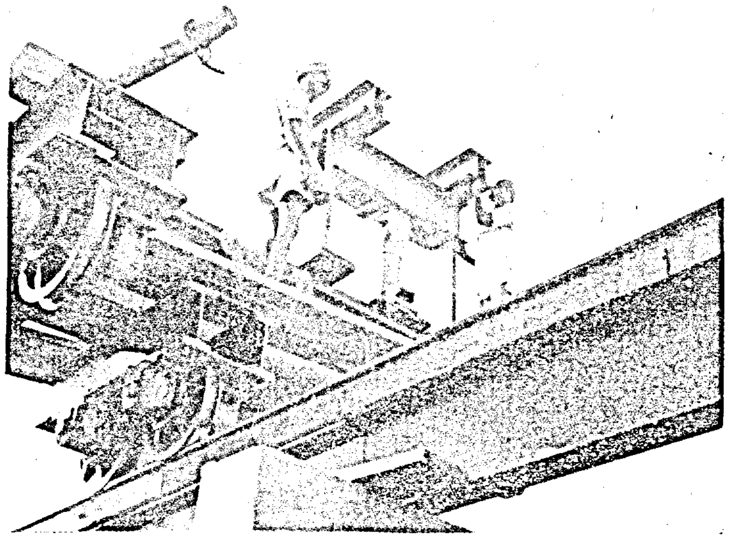


Photo 12. **Trolley from below**

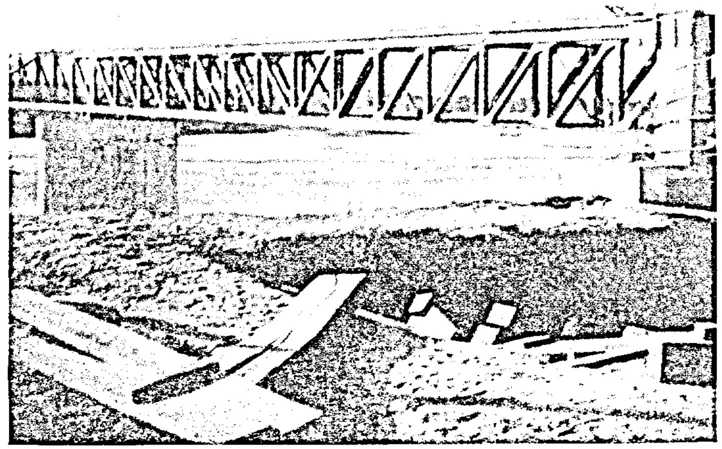


Photo 14. **Launching girder in position**

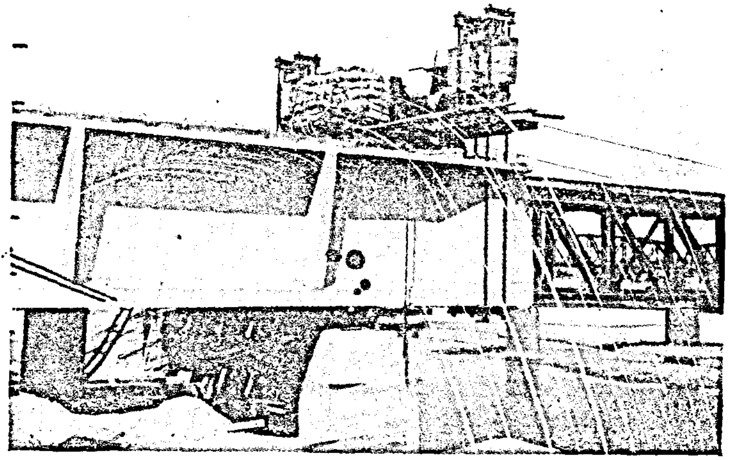


Photo 15. **Precast prestressed girders being launched**

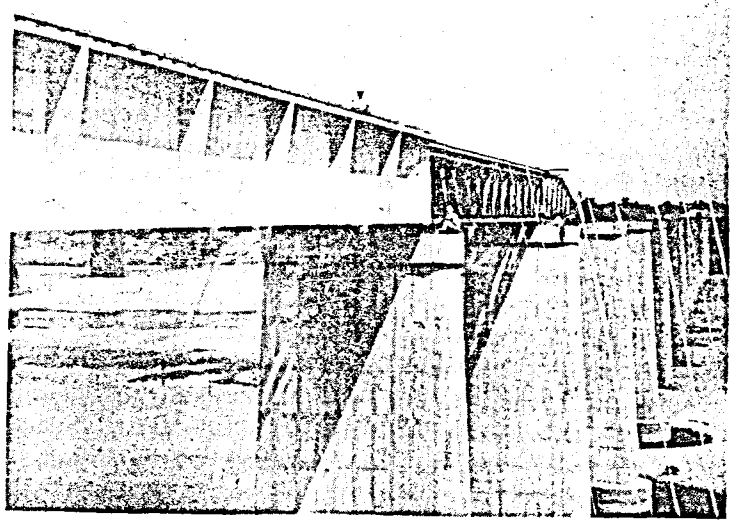


Photo 16. **Precast prestressed girders being launched**

(4) water regulating tank attached to each mixer for regulation of water-cement ratio, and

(5) five internal vibrators and two external vibrators.

The first two feet of the end blocks were done in rich mix 1 : 1 : 2, while the remaining length of the girder was concreted in 1 : 1.4 : 3 (by weight) high grade mix designed for the specified stress. On completing the concreting in layers up to the joint of the web top and the top flange, dowels were inserted, Photo 7, and concreting of the final layer was carried out keeping necessary castellations.

During concreting, eight concrete cubes at four intervals were taken for testing the crushing strength of the concrete at 3, 7, 14 and 28 days. This was necessary as it was specified that the stress at prestress shall be 4,000 lb per sq. in. and the strength at 28 days shall be 5,000 lb per sq. in. The concrete test cubes were cured under the same conditions in which the girder was cured by keeping them on top of the girder after removal of shuttering. Test results showed that at 7 days the strength attained was about 5,000 lb per sq. in. and as such prestressing could have been carried out at 7 days. However, to be on the safe side, prestressing was carried out after 10 days.

7.4. Removal of Side Moulds

To avoid disruption of the concrete it was essential to remove the side moulds of the girder as early as possible and before shrinkage of the concrete started. A model sample of the girder piece was prepared of the same mix and it was found that at the earliest the moulds could be removed after nine hours without disturbing the concrete. Accordingly the side moulds were removed after nine hours.

The girder was then covered with gunny cloth which was kept wet and curing was done for 14 days. Necessary washers and nuts were threaded on to the projecting ends of the bars and the uncovered threaded portion of the bars was kept well protected by a cap so that no thread was damaged. Threads on these bars are of a special type, Plate VI, and it was essential to safeguard against any damage.

7.5. Prestressing Operation

Equipment : (1) Lee McCall 42 ton special jack

(2) A Tommy bar and ring spanner for tightening the nut as prestressing proceeded.

Before application of the Lee McCall Jack the Macalloy bar ends were attended to as follows:

(a) The nut at the bar was unscrewed and the end washer was taken out and threads were cleaned and dirt removed.

(b) Requisite number of Lee McCall washers were put at the ends such that the threaded length of the bar projecting from the washers was 4.61 in., the total threaded length being $7\frac{1}{8}$ inches. Macalloy bars are always cut to a tolerance of plus $\frac{1}{4}$ in. and the concrete girder is cast to a tolerance of minus $\frac{1}{4}$ in. and the arrangement has to be such that the Macalloy bar projecting from the end bearing plate should be 4.61 in. plus one 3/16 in. washer. If the bar happens to be somewhat longer than this, necessary extra washers to compensate for the additional length have to be inserted to bring the threaded length of bar to 4.61 in. beyond the washer surface. The point is very important as will be seen from Appendix 3 that the relative extension required for the given stress is 2.65 in. at each end. So when there is full relative extension of 2.65 in., the nut when tightened against the washer should have $(d/8=9/8\times 1/8)=0.14$ in. of unthreaded bar inside as necessary for the condition of high efficiency according to the Lee McCall specifications, Plate VI.

(c) On inserting the requisite washers the high efficiency nut was screwed on and tightened against the washer.

Prestressing was then carried out with the 42 ton jack specially designed for use with this system, Photo 8. Plate V gives the outline details of its parts. The jack was connected by means of a cotter to an adapter temporarily screwed on to the extended thread at the end of the bar, and the thrust was transmitted through the legs of the jack to the end anchor plate of the unit. The jack was then actuated by a hand operated pump and the relative extension of the bar read on the vernier, Photo 9 and could be checked against the force read directly on the pressure gauge. When the full stress had been applied to the bar as verified by the relative extension and dial reading, the anchor nut was screwed home and the jack disconnected. It is quite important to see that when the nut is tightened up, the gauge reading drops by 2 to 3 tons proving that the force is being taken up by the nut.

In this case, the total relative extension required was 5.3 in. whereas the jack run was $3\frac{1}{4}$ inches. Again, there were some curved bars to be stressed. Hence the stressing operation was conducted simultaneously and equally from both ends. This had also the advantage of reducing frictional effects.

To ensure synchronisation of the stressing operation from both ends the stressing was done in instalments of $\frac{1}{2}$ in. at each end and the pressure gauge readings were tallied at each such instalment. The results of stressing are given in Appendix 4.

In some bars the stressing was stopped at a relative extension of less than 2.65 inches. This was because as the stressing operation continued from one bar to another more and more elastic compression resulting in elastic strain was induced in the girder concrete. The relative extension for the subsequent bars was thus required to be reduced by the amount of shortening of the girder. The shortening of the girder was measured by counting the number of turns required for the nut to tighten against the end plate. There were 12 turns per one inch and, as such, for one turn of re-tightening, the calculated relative extension of 2.65 in. was required to be reduced to 2.57 and so on. Had the compression induced in the concrete been completely uniform as the stressing progressed from one bar to another, the last bar would have the minimum relative extension, but as will be seen from Appendix 4 it is less than 2.65 in. but not the minimum.

Before launching, thirteen out of fifteen bars were stressed as was necessary according to the design, otherwise the compressive stress in concrete would have exceeded that allowable. The remaining two bars in the centre were to be stressed after the deck slab was cast in situ. Grouting of the bars was usually carried out immediately after the full prestress was applied on thirteen bars in order to give the grout sufficient time for hardening before handling of the girder. The grouting was carried out by a combined grout mixer and pump. Grouting was done through the slot in the end plate connected to the annular space as shown in Plate VI. Macalloy bar diameter is $1\frac{1}{8}$ in. and the internal diameter of the unitube sheath is $1\frac{1}{2}$ in. Thus the annular space to be filled with grout was very small and it was imperative to see that the cavities were completely filled up by grouting from one end until the grout got expelled from the other end. To ensure this further tell-tales were also kept for every bar in the centre. The grout mix was one of cement to two of sand.

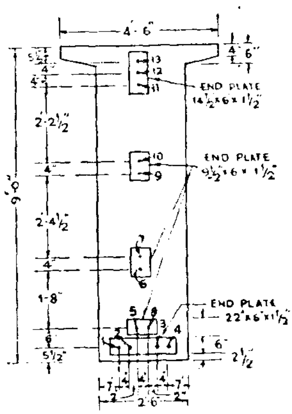
After all the operations were over, the projecting ends of the Macalloy bars were cut off with a hacksaw of high speed level.

The whole operation of prestressing 13 bars took about 4 hours. So in one day of eight working hours, two girders could be prestressed.

The stressing sequence of bars was as follows:

10, 9, 7, 6, 12, 5, 8, 11, 13, 2, 3, 1, 4.

It was essential to follow the sequence so as to prevent unsymmetrical stresses.



Quantities for one girder

Concrete	1,400 cu. ft.
Cement	17 tons
Macalloy bars	$2\frac{1}{2}$ tons
Secondary steel	2 tons

8. LAUNCHING

8.1. The precast beams weighing 95 tons each posed the difficult problem of transporting them from the casting yard to their final position on the pier bearings. The equipment used consisted of two latticed girders 182 ft long and 9 ft deep each, braced together at 6 ft 6 in. centres, to span two piers with one 10 ft bay overhanging towards the rear where the prestressed girder is picked up, and 56 ft of light weight nose overhanging at the forward end. Rails were fixed on to the top chords of the launching girder and two trollies run on these, Plate VII. Under the trollies another pair of wheels was fixed at 13 ft 6 in. centres. This outside pair of wheels was used to run the trolley over already placed prestressed girders when the launching girder had moved to a subsequent span. In addition, the trollies carry a platform with a jack on each side. The cross beams of the trolley were designed to carry one prestressed beam suspended on each side. Plummer blocks were provided for running the launching girder from span to span. The launching was done in stages.

8.2. First Stage

The launching girder was assembled on level ground directly behind the abutment and was run out to the first span. A rail track 13 ft 6 in. centre to centre supported on pillars was made for the trolley line in the yard up to the first pier, Photo 10. One trolley was put on the track and fixed rigidly to the rear end of the launching girder. A wire rope tied to the nose of the launching girder was tied to the winch fixed on the end pier on the other side of the river. As the rope was being wound, the girder moved and the trolley was holding the end of the launching girder so that the girder could roll smoothly over the plummer blocks fixed 6 ft 6 in. centres, on the first two piers. The hauling was further helped by fixing two winches on the trolley and passing the two ropes on to the third pier and fixing it. So when these two winches were winding the wire rope the trolley was being assisted in its motion and the run was quite smooth and straight.

The launching girder, shown in outline in plate VIII was so designed as to be stable when the launching nose just touched the next pier.

After the launching girder came into position across the first span, the girder end on the yard side was jacked up and the plummer blocks removed. The end was put back in original position on packings in place of plummer blocks and the plummer blocks were taken to the third pier and fixed in position.

8.3. Second Stage

The trolley fixed with the launching girder was then freed and brought back to the yard on the temporary track after leaving the launching girder in position. A second trolley was then put on the rail track in the yard and kept on the side of the other end of the precast girder in the yard. When both the trollies were in their respective positions ready to pick up the girder, two jacks of 50 ton capacity each were mounted on each trolley for picking up the precast girder suspending the same by means of four $2\frac{1}{2}$ in. rods as shown in Plate VII. The casting of the girders had been done symmetrically on both sides of the trolley line. This naturally involved sideway movement of the girders to enable the trolley to pick them up. The method adopted for effecting this sideway movement is given below:

(1) The girder was jacked up at both ends by means of 60-ton capacity jacks.

(2) A channel, a row of rollers, and an R. S. joist piece with necessary packing were inserted in between the support and the bottom of the girder.

(3) By means of chain and pulley at both the ends, movements was effected towards the rail track. The rolling friction was small and an effort of only about $1\frac{1}{2}$ tons on each side was enough to move the girders weighing about 95 tons each.

After picking up two girders and placing them on the two trolleys, one at each end, the first trolley was moved by working the winches mounted on it. This involved the movement of the other trolley also as it was connected with the first through the prestressed girders.

8.4. **Third Stage**

When the first trolley, while moving came to the overhanging portion of the launching girder, the motion was slowed down as the movement was now to be taken on the inner wheels of the trolley and to the launching girder. The 10 ft overhang in launching girder was thus necessary for transfer of girder loads without shock. The level of the top rails fixed on the launching girder was slightly higher so that when the inner wheels of the trolley got engaged on the rails fixed on the launching girder, the wheels on the outer track were off the rails. Two pieces of rails tapered both vertically and horizontally were inserted at the end for easy transfer. Thus the trolly carrying the prestressed girders on both sides moved on to the launching girder until the girders were exactly in correct longitudinal position in relation to their bearings.

8.5. **Fourth Stage**

As stated above, the precast girders in their turn formed the support for the track extension, the girders held in position by the trolley lines remaining sufficiently on one side of their final alignment. A worm and screw arrangement fitted to the cross girders of the trolley lines was required to be operated before the girders were lowered and put in position on the bearings.

8.6. **Fifth Stage**

When the prestressed girders were in position the first trolley was again fixed to the rear end of the launching girder and it was made to move on the next span.

The trollies then came back and picked up another pair of precast girders and carried them over to the next span, the track being extended over the precast girders already in place on the piers. This process was repeated for launching all the remaining girders, Photos 12 to 16. The various stages of launching are shown diagrammatically in Plate VIII.

The whole operation of launching one pair of girders took on an average five days.

9. IN SITU WORK

9.1. In situ work consisted of casting the diaphragm between the girders and laying the deck slab.

9.2. **Diaphragm between Girders**

Details of shuttering will be as shown in Plate IX. It is proposed to support the shuttering on the bottom flange of girders. Shuttering will be of mild steel plates stiffened with angle irons. Necessary holes for fixtures have been kept in girder flanges and ribs during concreting of the girder. The diaphragms are proposed to be concreted in advance of the deck slab with 1 : 2 : 4 concrete mix.

9.3. **Deck slab**

The shuttering for the deck slab is proposed to be supported on the girder web as shown in Plate IX. Necessary holes for the purpose of fixing angle bearers and verticals have been provided in the girder web during concreting of the girder. The erection and removal of shuttering is proposed to be done by means of a mobile suspended platform. Details are shown in Plate IX. This platform will move on trolleys over rails fixed on the top of girders to carry materials for erection of the centering. After the slab is cast and cured the same platform will run on rails to be fixed on the slab for removal and taking the shuttering to other spans and so on. In order to avoid the obstruction of pier and for through movement of the platform, hinges are provided on the channel bolted to the underframe.

It is proposed to use three such platforms for concreting the sixteen spans of 110 ft each. It is expected that about four spans will be concreted in a month and it will take four months to complete the job.

The slab will be of high grade cement concrete with a working stress of 950 lb per sq. in. The concrete shall be vibrated with pin and screed vibrators. Necessary expansion joints will be provided at every 110 ft over the piers.

The thickness and reinforcement details of the slab will be as per Plate IV.

10. BEARINGS

10.1. The fixed and free bearings are of cast steel conforming to the Provisional I. R. C. Standard Specifications.

10.2. **Free Bearings**

The top plate of the bearing was fixed with the girder while casting. The bottom plate was fixed in the pier cap with the necessary bolts etc. The roller was then inserted and kept in position by a removable frame and the girder was slowly lowered in position just to fit the lug into the recess of the roller.

10.3. Fixed Bearings

On fixing the lower plate of the bearing on the pedestal on pier cap, a pin 1 in. diameter and 1½ in. long was inserted and the girder was lowered so as to fit in the pin. The top plate was fixed to the bottom of the girder while casting.

11. MATERIALS

11.1. Macalloy Steel Bars 1⅛ in. Diameter

The alloy steel contains :

	Per cent
Carbon	0.53 to 0.65
Silicon	1.70 to 2.00
Manganese	0.70 to 1.00
Sulphur	0.018 to 0.03
Phosphorus	0.026 to 0.031

After being specially treated it has an ultimate tensile strength from 64 to 72 tons per sq. in. and 0.2 per cent proof stress of over 58 tons per sq. in. Sixteen samples of these Macalloy bars were machined in the middle portion so as to reduce the diameter to about 0.98 in. and were tested in the V. J. T. Institute, Bombay. Results are given in Appendix 5. The average strength of the bar worked out to be 67.5 tons per sq. in. The stress strain curve shown in Plate X is practically straight up to well over the initial prestress of 42 tons per sq. in. which worked out to about 0.65 of 64 tons per sq. in. For this particular alloy, tests show that there is practically no creep.

Rods are available in this quality steel up to a diameter of 1⅛ in. The ends of the bars are provided with special form of screw thread to take the high efficiency nuts which when fully tightened against the ends of the concrete member maintain the load on the rods.

11.2. High tensile rods suitable for prestressing have no yield point of the kind that characterises mild steel. The term "equivalent yield stress" or "0.2 per cent proof stress" used for

this high tensile steel denotes the stress at the point of the stress strain curve at which the plastic or residual strain equals 0.002. It will be seen from Plate X that the elastic range of rods extends practically to a stress larger than the initial prestress.

Initial working stress allowable is 42 tons per sq. in.

11.3. Sheath of Steel Unitubes

In a curved tendon (bar) the percentage loss of stress depends directly on the coefficient of friction between the materials involved. The value of the coefficient for high tensile steel on concrete is about twice that for high tensile steel on steel. The Macalloy bars are therefore inserted into steel unitubes and kept in position before casting the girder. These tubes are left in the girder, and the annular space between the tubes and the bars is grouted after prestressing. Further to reduce the friction, these tubes are lead coated on the inside. The diameter of these tubes is 1⅜ in. as against 1⅛ in. of the rods. Tubes are available in coils imported from Europe along with Macalloy bars. The jointing of these tubes is done by tape.

The estimation of friction is given in Appendix 6

11.4. The specified stresses for the concrete are as follows :

	lb per sq. in.
Concrete cube crushing strength at 28 days	5,000
Safe working stress	1,500
Stress at transfer	4,000

To attain the above stresses, concrete mix of the following proportions was adopted :

	lb
Cement	168
Sand	200
Crushed stone grit black trap 3/16 to ⅜ in.	130
Crushed stone black trap ⅜ to ¾ in.	140
Shingle ⅜ to ¾ in.	260

The water-cement ratio was 0.40, i.e. 4½ gallons of water per one bag of cement weighing 112 lb. The mix was found to give satisfactory test result.

11.5. Fine Aggregates

The Mahi river sand is fine and so coarser particles were required to be added to it to bring the sieve analysis curve within the standard curves for fine aggregate, Plate X. Crushed black trap metal grit 3/16 to $\frac{3}{8}$ in. from the quarry at Timba was added in suitable proportion.

11.6. Coarse Aggregate

This comprised the following :

(1) Mahi river shingle $\frac{3}{8}$ to $\frac{3}{4}$ in.

(2) Crushed stone (black trap) 3/16 to $\frac{3}{4}$ in.

(3) Crushed stone grit (black trap) 3/16 to $\frac{3}{8}$ in.

Mahi river shingle is mostly quartz and useful for the purpose. The maximum size of coarse aggregate to be used was specified as $\frac{3}{4}$ in. Shingle of $\frac{3}{8}$ to $\frac{3}{4}$ in. size was not readily available in Mahi river as it was mixed with kankar nodules which had to be hand picked. So, to supplement this, crushed stone (black trap) kapachi $\frac{3}{8}$ to $\frac{3}{4}$ in. available from Timba quarry near Sewalia was required to be added in suitable proportion. The coarse aggregate composed of Mahi river shingle and crushed stone kapachi from Sewalia, however, lacked in smaller sizes. below $\frac{3}{8}$ in. Crushed stone grit (black trap) 3/16 to $\frac{3}{8}$ in. was, therefore, added to suit the grading curve, Plate X.

The whole mix works out to 1 : 1.4 : 3 approximately by weight and per 100 cu. ft. of concrete 24 bags of cement weighing 112 lb each were used.

Plant and machinery used in the work are listed in Appendix 7.

12. COSTS

			Rs
12.1.	Cost of substructure with foundations		14,51,000
	Total cost of superstructure as per accepted tender (excluding pitching approaches and box returns)		11,13,912
	Ratio of cost of substructure to superstructure	1.3	

12.2. Cost of Bridge

		Rs
1.	Per sq. ft. of carriageway	82
2.	Per linear foot of bridge	2,000
3.	Per sq. ft. of elevation above bed level	25
4.	Per cu. ft. up to the lowest level of foundation without deduction for span opening	-/10-
5.	Per cu. ft. of bridge of actual construction	6
	Total likely expenditure on the whole bridge including box returns, approaches, pitching and W. C. Establishment, etc.	34,59,000

ACKNOWEDGMENT

The Authors are thankful to the facilities given by the Hindustan Construction Company in making free use of their working drawings and design calculations. They are also thankful to Shri V. N. Parikh, Deputy Engineer, Mahi Bridge Sub-division, for the assistance in the compilation of the data and checking up of calculations.

Appendix 1

COMPARISON OF COSTS OF SUPERSTRUCTURE (TWO-GIRDER SYSTEM) FOR (1) R.C.C. BALANCED CANTILEVERS, AND (2) PRESTRESSED GIRDERS

R. C. C. Two-Girder System		Prestressed Two-Girder System		Saving	
Item	Quantity cu. ft	Item	Quantity cu. ft	Quantity	Amount Rs
Concrete of girders and cross girders (1 : 2 : 4)	54,400	Concrete for girders * (special grade)	44,800	6,600 cu. ft	66 × 750 = 49,500
		Diaphragms (high-grade)	3,000		
Concrete for deck slab (1 : 2 : 4)	31,350	Concrete for deck slab (high grade)	31,350		
Total	85,750 cu. ft.		79,150 cu. ft		
Cement	772 tons	Cement	856 tons	-86 tons	86 × 100 = - 8,600
Mild steel for girders and slab	608 tons	Mild steel	190 tons	418 tons	418 × 700 = 292,600
		High tensile steel	80 tons	-80 tons	80 × 2000** = - 160,000
			270 tons		
				Total Saving:	Rs 1,73,500

* Special grade concrete requires 24 bags per 100 cu. ft. High grade concrete requires 18 bags per 100 cu. ft.

** Rs. 2,000 per ton includes anchorages etc. complete.

Appendix 2

DESIGN CALCULATIONS OF THE GIRDERS

1. The properties of the girder are given in Table 1. The section of the girder and the composite section are given in Fig. 1 and 2.

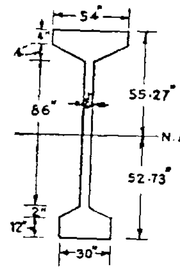


Fig. 1.

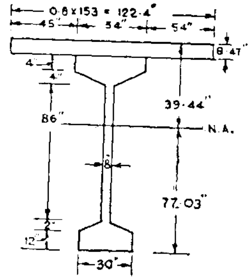


Fig. 2.

TABLE 1

Properties of the Girder

	Girder without slab	Girder without slab but after prestressing 13 bars and grouting Fig. 1.	Girder with slab (Composite section) Fig. 2.
1. Area in sq. inches.	1399.5	1474.167	2522.446
2. Moment of inertia about the neutral axis in.[4]	2147178	2315459	4514786
3. Distance of neutral axis from top in.	52.80	55.27	39.44
4. Distance of neutral axis from bottom in.	55.20	52.73	77.03
5. Section modulus $Z_t = I/Y_t$ in.^{3}	40667	41892	145780
6. Section modulus $Z_b = I/Y_b$ in.^{3}	38899	43910	58611

In all there were 15 Macalloy bars out of which 13 bars were stressed and grouted first and the remaining two bars were stressed after casting the slab for keeping the initial stress within permissible limits.

2. Details of loading and bending moments for the prestressed girder are given in Table 2.

TABLE 2

Bending Moments for Class A Loading

Sl. No.	Item	Dead load per linear ft. lb	Maximum B.M. on girder in.-lb	Remarks
1	Girder ...	1486	28.75×10^6	The B.M. to be resisted by girder alone.
2	Stiffener ...	140		
3	Castellation ...	48.3		
4	Slab (less Castellation)	1301.7	24.293×10^6	To be taken by girder alone as the slab centering is supported by the girder.
5	Diaphragms ...	112.9		
6	Surfacing including kerb and parapet etc.	559.5	9.61×10^6	To be resisted by the composite section.
7	Live load Class A loading, 2-lanes ...		38.28×10^6	
			100.93×10^6	

3. Stress in top and bottom fibre of the girder at different stages of loading:

(1) Stresses due to prestress

Initially 13 bars were stressed to 42 tons per sq. in. giving total —

Initial force, $P_{13} = 13 \times .994 \times 42 \times 2240 = 1215702$ lb say.

Moment due to $P_{13} = P_{13} \times e_{13}$ where e_{13} is the distance of centre of gravity of 13 bars from the centre of gravity of the girder.

This comes to $1215702 \times (55.2 - 6.54) = 59.16 \times 10^6$ lb-inches.

Therefore

f_t = stress at the top fibre of the girder

$$= \frac{P_{13}}{A} - \frac{(P_{13} \times e_{13})}{Z_t}$$

$$= \frac{1215702}{1399.5} - \frac{59.16 \times 10^6}{40667}$$

$$= (868 - 1455) = \text{minus } 587 \text{ lb per sq. in.}$$

f_b = stress at the bottom fibre of the girder

$$= \frac{P_{13}}{A} + \frac{(P_{13} \times e_{13})}{Z_b}$$

$$= \frac{1215702}{1399.6} + \frac{59.16 \times 10^6}{38899}$$

$$= 868 + 1521 = 2389 \text{ lb per sq. in.}$$

(2) Stresses due to dead load of girder etc.

M_{dl} = moment of girder plus stiffener plus castellation

$$f_t = \frac{M_{dl}}{Z_t} = \frac{28.75 \times 10^6}{40667} = 707 \text{ lb per sq.in.}$$

$$f_b = \frac{M_{dl}}{Z_b} = \frac{28.75 \times 10^6}{38899} = \text{minus } 740 \text{ lb per sq.in.}$$

(3) Stresses due to dead load of deck slab etc.

M_{ds} = moment due to dead load of deck slab etc.

$$f_t = \frac{M_{ds}}{Z_t} = \frac{24.293 \times 10^6}{41892} = 5801 \text{ lb per sq. in.}$$

$$f_b = \frac{M_{ds}}{Z_b} = \frac{24.293 \times 10^6}{43910} = \text{minus } 553 \text{ lb per sq.in.}$$

(4) Stresses due to prestressing of remaining two bars

Prestressing force, $P_2 = 2 \times 0.994 \times 2240 \times 42 = 18703$ lb

Moment $P_2 \times e_2 = 187031 \times (77.03 \text{ minus } 19.5)$

$= 10.76 \times 10^6$ in.-lb.

Therefore $f_t = \dfrac{P_2}{A}$ minus $\dfrac{(P_2 \times e_2)}{Z_t}$

where Z_t = modulus of section and A is area of composite section.

$$= \frac{180731}{2522.446} - \frac{10.76 \times 10^6}{145780}$$

$$= 1 \text{ lb per sq. in.}$$

$$f_b = \frac{187031}{2522.446} + \frac{10.76 \times 10^6}{58611}$$

$$= 259 \text{ lb per sq. in.}$$

(5) Stresses due to dead load of finishes etc. and live load

M_1 = moment due to surface finishes etc. and live load

$$f_t = \frac{M_1}{Z_t} = \frac{47.89 \times 10^6}{145780}$$

$$= 329 \text{ lb per sq. in.}$$

$$f_b = \frac{M_1}{Z_b} = \frac{47.89 \times 10^6}{58611}$$

$$= 818 \text{ lb per sq.in.}$$

The stresses have been summarised in Table 3 and shown in Fig. 3 and 4. Fifteen per cent loss of prestress due to creep etc. in the concrete has been assumed.

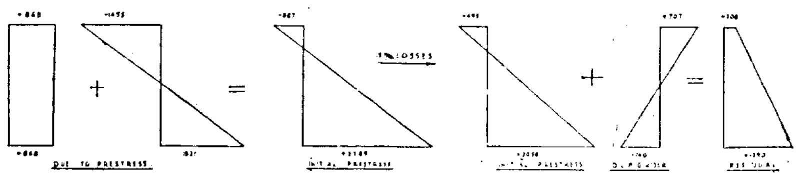


Fig. 3.

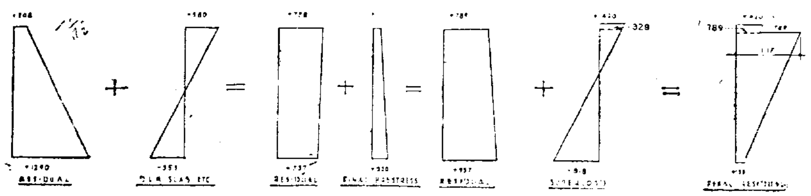


Fig. 4.

TABLE 3

Stresses

Girder Alone

	Due to Prestress	Residual 15 per cent less	Dead load moment of girder etc.	Residual	Dead load moment of slab etc.	Residual
F_t	–587	–499	707	208	580	788
F_b	2389	2030	–740	1290	–553	737

Composite Section

	Residual	Final prestress	After loss of 15 per cent	Residual	Live load moment	Final stress
F_t	788	1	1	789	329	1118
F_b	737	259	220	957	–818	139

(4) Stress limitations for the design

The stress limitations for the design are as follows :

(1) Concrete in the girder to have a minimum compressive strength of 4,000 lb per sq. in. when the prestressing is done and 5,000 lb per sq. in. at 28 days (cube crushing strength).

Maximum initial working stress in concrete shall be equal to 1,500 lb per sq.in.

(2) $1\frac{1}{8}$ in. diameter Macalloy bars to be stressed to 41.7 tons giving a stress of 42 tons per sq.in.

Stresses as seen from the stress diagram are within permissible limit at every stage.

(5) Factor of Safety at first crack

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

(1) Moment at first crack $M_{cr} = M_t$ plus $f_{cr} \times Z$. Where M_{cr} is moment at which first crack is anticipated in concrete. M_t is bending moment due to all dead and live loads. f_{cr} is allowable stress causing first crack in concrete at the bottom fibre.

Now, first crack will appear when total tensile stress in bottom fibre is 500 lb per sq. in. M_t causes a compressive stress of 139 lb per sq. in. Hence total allowable stress to cause first crack in the bottom fibre is given by

f_{cr} = minus (139 plus 500) = minus 639 lb per sq. in.

Z_b of composite section = 58611 in.^{3}

Therefore the additional moment which can be taken up by the section for the first crack

$= 58611 \times 639$

$= 37 \times 10^6$ in.-lb.

M_{cr} = (100.93 plus 37) $\times 10^6$ in.-lb.

i.e. against the total moment there is factor of safety of 1.37.

Live load moment without surface finishes etc. is 38.28×10^6 in.-lb

$M_{cr} = 137.93 \times 10^6$ in.-lb.

$M_{dl} = 62.65 \times 10^6$,,

$M_{ll} = 38.28 \times 10^6$,,

Therefore factor of safety with reference to live load only will be

$$\frac{M_{cr} - M_{dl}}{M_{ll}}$$

$$= \frac{137.93 - 62.65}{38.28}$$

$$= \frac{72.28}{38.28}$$

= 2 (Approximately)

(6) Investigation of Section at Ultimate Load

For ultimate load investigation, effect of prestress is assumed to be nullified due to plastic elongation of steel causing a stressless plastic strain equal to the elastic strain originally required to maintain the requisite prestress.

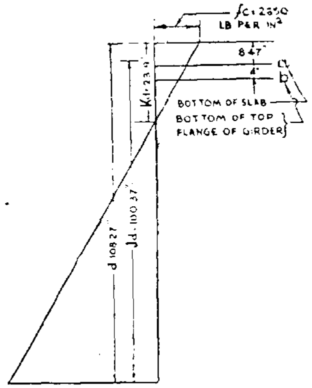


Fig. 5.

Concrete in the bottom fibre has already failed due to tension and only the section above neutral axis is effective.

Hence the beam is treated as a conventional reinforced concrete member subjected to simple bending moment, Fig 5.

Tensile stress in concrete is disregarded.

f_c is ultimate compressive strength of 2850 lb per sq. in. in the slab of high grade concrete.

f_t is ultimate tensile strength of 145000 lb per sq. in.

e is strain in the extreme fibre in concrete i.e. 0.003

Effective modulus of elasticity

$$= \frac{2850}{.003} = 0.95 \times 10^6 \text{ lb per in.}$$

$$n \text{ i.e. modular ratio} = \frac{\text{Modulus of elasticity of steel}}{\text{Modulus of elasticity of concrete}}$$

$$= \frac{25 \times 10^6}{0.95 \times 10^6} = 26.6$$

p i.e. percentage of steel

$= \dfrac{A_s}{bd}$ where bd is the effective area in sq. in.

$$= \frac{15 \times 0.994}{2324} = 0.0062$$

fc = 2850 lb per sq. in.

$$k = pn + \sqrt{p^2\, n^2 + 2\, pn}$$
$$= 0.006 \times 26.6 + \sqrt{(0.006 \times 26.6)^2 + 2 \times 0.006 \times 26.6}$$
$$= .1596 + \sqrt{0.0254 + 0.319}$$
$$= 0.159 + .058$$
$$= 0.217$$
$$= \text{say } 0.22$$
$$kd = 0.22 \times 108.27 = 23.9 \text{ in.}$$
$$jd = 0.93 \times 108.27 = 100.37 \text{ in.}$$

stress at a, Fig. 5 $= 2850 \times \dfrac{15.43}{23.90} = 1850$ lb. per sq. in.

stress at b, Fig. 5 $= 2850 \times \dfrac{11.43}{23.90} = 1360$ lb per sq. in.

To find the resisting moment of the concrete section above neutral axis it is necessary to know whether the composite unit will fail in concrete or steel and to find out the effective ultimate resisting moment.

Resisting moment of force on slab

$$= \frac{(2850 + 1850)}{2} \times 122 \times 8.47 \times 104.04$$
$$= 255 \times 10^6 \text{ in.-lb.}$$

Resisting moment of force on top flange of girder

$$= \frac{(1850 + 1360)}{2} \times 54 \times 4 \times 97.80$$
$$= 0.34 \times 10^6 \text{ in.-lb.}$$

Resisting moment of force on web

$$= \frac{1360}{2} \times 8 \times 11.43 \times 88.18$$
$$= 5.5 \times 10^6 \text{ in.-lb.}$$

Thus total moment in concrete section with top fibre having stress of 2850 lb per sq. in. will be 260.84×10^6 in.-lb.

The resisting moment in steel M_u is also equal to $A_s \times f_u \times ja$

where A_s is area of steel Macalloy bars, jd is lever arm and f_u is ultimate stress.

$$M_u = 15 \times 0.994 \times 66 \times 2240 \times 100.37$$
$$= 220 \times 10^6 \text{ in.-lb.}$$

Thus the effective resisting ultimate load moment will be 220×10^6 in.-lb and the girder will fail in steel since before the stress in concrete reaches 2850 lb per sq. in., the steel would reach its ultimate stress of 66 tons per sq. in.

$$M_u = 220 \times 10^6 \text{ in.-lb.}$$
$$M_{dl} = 62.65 \times 10^6 \text{ in.-lb.}$$
$$M_{ll} = 38.28 \times 10^6 \text{ in.-lb.}$$

The factor of safety with reference to total load will be

$$\frac{M_u}{M_{dl} + M_{ll}} = \frac{220 \times 10^6}{100.37 \times 10^6} = 2 \text{ approximately.}$$

The factor of safety with reference to only live load will be

$$\frac{M_u - M_{dl}}{M_{ll}} = \frac{(210 - 62.65) \times 10^6}{38.28 \times 10^6}$$
$$= \frac{147.35}{38.28}$$
$$= 4 \text{ approximately.}$$

(7) Shear Calculations

1. Dead load shear

Dead load of girder etc. = 1674.3 lb per rft.
,, of slab etc. = 1414.6 ,,
,, of surface etc. = 559.5 ,,
3648.4 lb ,,

Dead load shear $= 3648.4 \times \dfrac{107}{2} = 195189$ lb.

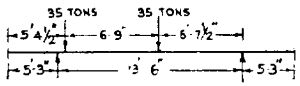


Fig. 6.

2. Live load shear, class AA loading

$$R_A = \frac{W\,(6' - 7\tfrac{1}{2}'' + 13' - 4\tfrac{1}{2}'')}{13' - 6''}$$

$= 1.481\ W$

with impact allowance of

10 per cent $R_A = 1.481\ W + 0.148\ W$

$= 1.629\ W.$

i.e. $R_A = 1.629 \times 35 \times 2240$

$= 127713$ lb.

$\therefore$ Shear force $= \dfrac{127713}{107} \times 101$

$= 120551$ lb.

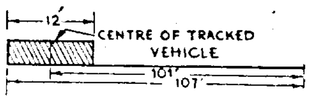


Fig. 7.

(8) Effect of bent up bars on shear at ends

Out of 15 bars at the centre, 8 bars have been taken straight in bottom flange and 5 are bent up at the end and two are bent up in the middle as shown in Plate IV. Details of 5 bent up bars are as shown in Fig. 8 and 9.

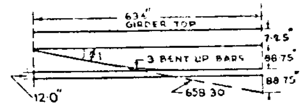


Fig. 8.

$$\text{Sin } \phi_1 = \frac{2 \times 88.75}{658.30}$$

$= 0.27$

Negative shear due to tensioning of three bent up bars.

$= 3 \times 42 \times 2240 \times 0.994 \times 0.27$

$= 75747$ lb.

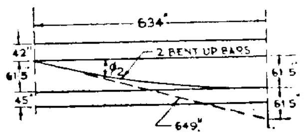


Fig. 9.

$$\text{Sin } \phi_2 = \frac{2 \times 61.5}{649} = 0.19$$

Negative shear due to tensioning of two bars bent up

$= 2 \times 42 \times 2240 \times 0.994 \times 0.19$

$= 35536$ lb.

Combining the above two total negative shear is 111283 lb.

After 15 per cent loss

Negative shear $= 0.85 \times 111283$

$= 94590$ lb.

Maximum unit shear at centroid is given by

$$s = \frac{S.\ A.\ \overline{Y}.}{b.\ I}$$

Where S = Horizontal shearing stress at centroid :

S = Total vertical shear on section.

b = Width of section at centroid.

I = Moment of Inertia of entire section about the centroid.

A = Area of section on either side of the centroid.

$\overline{Y}$ = Distance between the centre of grativity of this area and the centroid of section.

$A\ \overline{Y}$ of girder alone above neutral axis of composite section.

$= 8 \times 30.97 \times 15.49 + 46 \times 4 \times 28.97$

$+ \dfrac{46 \times 4}{2} \times 25.64$

$= 38378 + 53305 + 2358.9$

$= 11527.2$ in.^{3}

$A\ \overline{Y}$ of slab above neutral axis of composite section

$= 122.4 \times 8.47 \times 35.20$

$= 36492.8$ in.^{3}

Total $\overline{A\ Y} = 48020$ in.^{3}

Shear intensity on girder alone

Loads (a) 1674.3 girder etc.

(b) 1414.6 slab etc.

3088.9 lb per rft.

Shear $= 3088.9 \times \dfrac{107}{2}$

$= 165256$ lb.

$$Q = \frac{165256 \times 11527.2}{8 \times 2315459} = 103 \text{ lb per in.}^2$$

This intensity on girder alone will be negatived by the bent up bars

$$b_y = 103 \times \frac{94590}{165256} = 59.2 \text{ lb per in.}$$

Net intensity on girder $(q) = (103.0 - 59.2) = 43.8$ lb per sq. in.

Shear intensity on composite section.

Shear due to surfacing etc. $= 559.5 \times \dfrac{107}{2} = 29933$ lb.

Due to live load $= \dfrac{120551}{150484}$ lb.

Therefore $Q = \dfrac{150484 \times 48020}{8 \times 4514786}$

$= 200$ lb per sq. in.

Resultant shear stress $= 200 + 43.8 = 243.8$ lb per sq. in.

Principal tension $= \sqrt{\left\{ \dfrac{(f_c)^2}{2} + f_s^2 \right\}} - \dfrac{f_c}{2}$

$= \sqrt{434^2 + 243.8^2} - 434$

$= 63.8$ lb per sq. in.

Appendix·3

RELATIVE EXTENSION OF MACALLOY BARS

Relative extension in inches $X = 12\,L \sqrt{\frac{f_s}{E_s} + \frac{f_c}{E_c}}$

where L = length of member in feet equal to 109 ft (average).

f_s = stress in bar, in lb per sq. in. - 42 tons or 94,000 lb per sq. in.

f_c = stress in concrete adjacent to the bar, 1,500 lb per sq. in. allowable.

E_s = elastic modulus for Macalloy steel 25 × 10^6 lb per sq. in.

E_c = elastic modulus for the concrete assumed as 5 × 10^6 lb per sq. in.

$$\text{Then } X = 12 \times 109 \left\{ \frac{94000}{25 \times 10^6} + \frac{1500}{5 \times 10^6} \right\} = 5.30 \text{ in.}$$

So when the tensioning is started simultaneously at both ends, extension at one end will be $\frac{5.30}{2} = 2.65$ in.

The above calculations are based on the final compression in the concrete when all the bars have been tensioned. Thus to obtain the extension, the first bars to be tensioned will be slightly overstressed and this overstress will be successively reduced as stressing goes on. From the above it will be seen that the maximum elastic compression of concrete comes to

$$12 \times L \times \frac{f_c}{E_c}$$

$$= 12 \times 109 \times \frac{1500}{5 \times 10^6}$$

$$= 0.39 \text{ in.}$$

It means that the total relative extension shall vary from 5.30 to 4.91 in., i.e. 2.65 to 2.45 in. at one end.

APPENDIX 4

PRESTRESSING DETAILS FOR GIRDER No. 9

Bar No. in sequence	Baroda side								Vasad side								Average extension in inches on both sides	Average force in tons on both sides
	Initial reading on vernier	Final reading on vernier	Total extension	Reading on gauge giving total force exerted	Gauge Readings For extension in instalments				Initial reading on vernier	Final reading on vernier	Total extension	Reading on gauge giving total force exerted	Gauge Readings For extension in instalments					
					$\frac{1}{2}''$	$1''$	$1\frac{1}{2}''$	$2''$					$\frac{1}{2}''$	$1''$	$1\frac{1}{2}''$	$2''$		
10	0.57	3.22	2.65	45	10	19	29	37	0.46	3.11	2.65	44	10	21	28	36	2.65	44.50
9	0.63	3.28	2.65	45	11	20	28	37	0.91	3.56	2.65	44	11	20	29	36	2.65	44.50
7	0.72	3.37	2.65	44	10	20	27.5	35.5	0.88	3.53	2.65	44	11	19	28	36	2.65	44.00
6	0.83	3.48	2.65	44	10	19	27	35	1.01	3.66	2.65	45	10	20	28	36	2.65	44.50
12	1.87	4.42	2.55	45	12	21	30	38	1.22	3.79	2.57	44.50	9.5	18	28	35	2.56	44.75
5	0.70	3.27	2.57	45	11	20	29	37	0.97	3.58	2.61	43.00	10	19	26.5	35	2.59	44.00
8	0.94	3 43	2.49	45	12	21	29	37	0.53	3.07	2.54	43.50	9.5	19	26.5	35	2.51	44.25
11	1.13	3.70	2.57	44.5	11	20	29	37	0.15	2.69	2.54	45.00	10	19	27	35.5	2.55	44.75
13	2.82	4.35	2.53	45	12	21	30	38	0.21	2.75	2.54	43	9.5	18	27.5	35	2.53	44.00
2	0.73	3.38	2.65	44	11	19	25	36	0.65	3.32	2.57	43	10	20.5	29	37	2.61	43.50
3	0.91	3.40	2.49	42	10	20	28	34	0.53	3.10	2.57	45	12	21	28	37	2.53	43.50
1	0.88	3.37	2.49	42	10	19	27	36	0.80	3.37	2.57	43	9	18	26	34	2.53	42.50
4	0.92	2.45	2.53	43	10	19	28	36	0.32	2.89	2.57	42	10.5	19	26.5	35	2.55	42.50

Appendix 5

TEST RESULTS OF MACALLOY BARS*

S. No. of Bar	Ultimate tensile Stress. Tons per sq. in.	Second test for bar No. 10, 13 & 15. Tons per sq. in.	Percentage of elongation.
1.	68.64	—	9.6
2.	68.70	—	8.5
3.	69.04	—	10.0
4.	66.90	—	9.5
5.	67.84	—	8.3
6.	68.70	—	8.9
7.	68.44	—	9.8
8.	67.72	—	10.8
9.	68.00	—	8.1
10.	65.76	65.76	—
11.	69.26	—	9.5
12.	67.04	—	9.3
13.	64.90	64.64	10.8
14.	66.58	—	—
15.	63.50	64.30	—
16.	66.90	—	—

* (Carried out at V. J. T. Institute, Bombay.)

Appendix 6

ESTIMATION OF FRICTION IN PRESTRESSED GIRDERS

Due to the movement of the tendon relative to the concrete where tension is applied, friction comes into play. The existence of this friction is shown on site by a discrepancy between the force indicated by the gauge on the jack and the corresponding measured extension of the tendon on the vernier.

The total friction consists of

(a) friction in jack.

(b) friction in the anchorage if the tendon changes direction.

(c) friction due to any appreciable curvature of the tendon and

(d) friction due to movement of the straight portion.

In the Lee McCall system a loss occurs in the moving parts of the jack though there is no loss at the anchorage itself because of the straight pull on the bars. This loss is of the order of 3 per cent.

So when the dial registers 45 tons while tensioning the curved bar, force in the tendon at the jack equals to

$$0.97 \times 45 = 43.5 \text{ tons.}$$

Force in the tendon at the centre of the girder

In addition to the frictional losses *a* and *b*, mentioned above there will be losses (*c*) and (*d*) as enumerated above.

The estimation of losses due to (*c*) and due to (*d*) can be made as follows :

$$T = P \times e^{(Kx + u\phi)}$$

Where T = tension in the tendon at a distance x from the jack.

P = tension in the tendon at the jack.

e is the base of Napierian Logarithms.

ϕ is the total angle turned through by the tendon upto the point.

u and K are friction constant for Macalloy Bars. From the table of constants,*

$u = 0.30$ for duct of lead coated steel unitube

$K = 5^{-1} \times 10^{-4}$ for excess of duct size over bar size being ($1\frac{1}{2}'' - 1\frac{1}{8}'' = 3/8''$)

$$\phi = 15.7^\circ = \frac{15.7 \times \pi}{180} \text{ radians.}$$

Then

$$T = 43.5 \times e^{-\left(\frac{1}{5\times10^4} \times 55 + \frac{\cdot30 \times 15\cdot7 \times \pi}{180}\right)}$$

$$= 43.5 \times e^{-(.0011 + .081)}$$

$$= 43.5 \times \left(\frac{1}{2.718}\right)^{.08}$$

$$= \frac{43.5}{1.08}$$

$$= 41 \text{ tons approximately.}$$

From the Appendix 3 it will be seen that the extension of 5.30 in. is calculated on the force of 41.7 tons in the Macalloy bar. The difference of .7 tons between the two theoretical calculations might be due to estimation of frictional constants.

In practice the force applied is calculated and based on extension read on the vernier and not solely on the dial reading. The dial reading is simply taken for a rough check.

The theory of the friction estimation is roughly alright and thus it seems that about 3.3 tons of force is lost in friction bringing the friction loss to about 7.3 per cent.

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* Indian Concrete Journal, January 1955—page 33.

Appendix 7

PLANT AND MACHINERY USED FOR THE BRIDGE

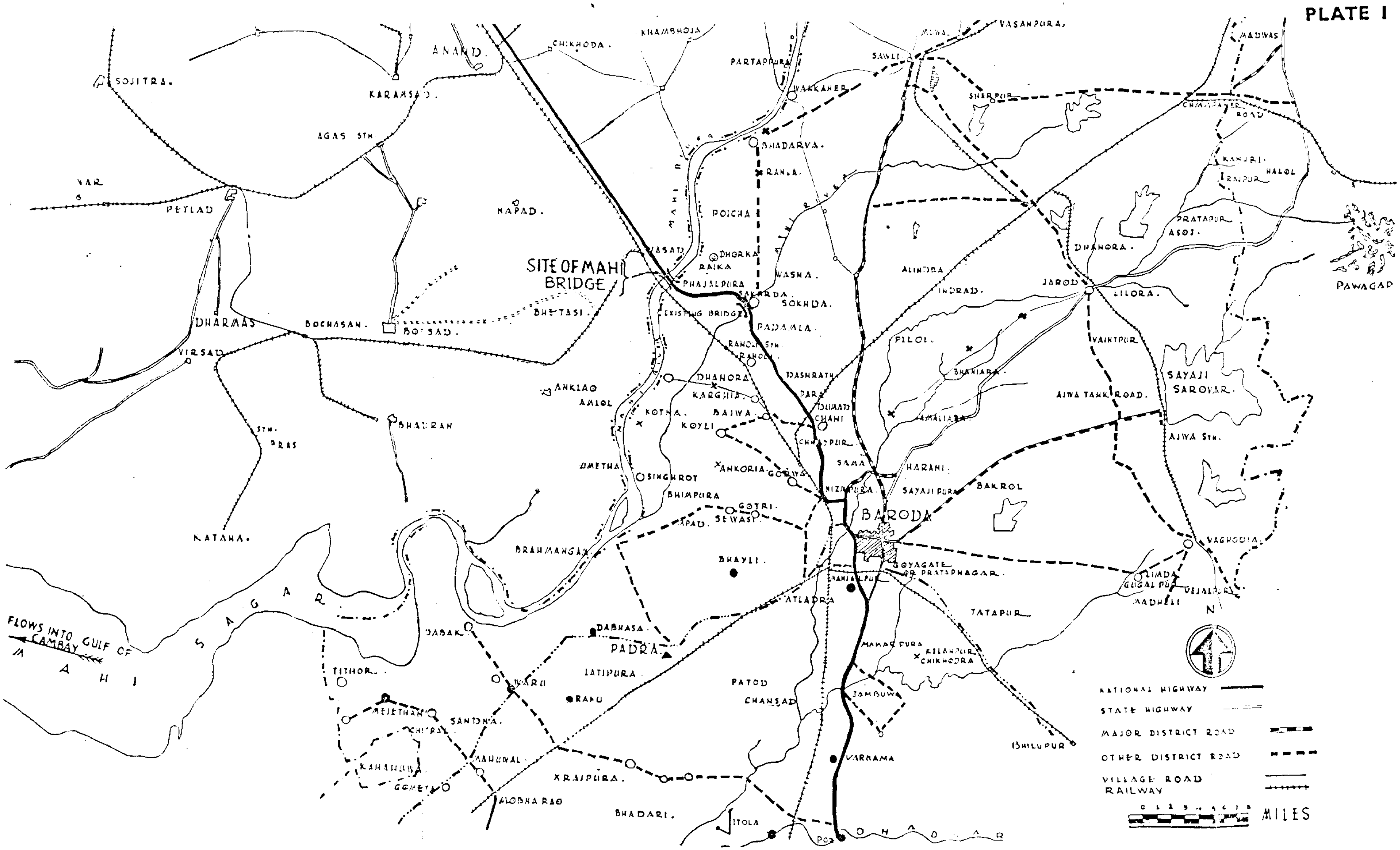
I. For Substructure

		No.
(1)	G. M. generating set 50 kw	1
(2)	Centrifugal pumps 8 in. × 6 in. to 3 in. × 2 in.	4
(3)	Sump pump $2\frac{1}{2}$ in.	1
(4)	Steam winches	2
(5)	Steam cranes	..
(6)	Gravel crusher	1
(7)	Compressor unit	1
(8)	Pnumatic paving breaker	4
(9)	Diving set with helmet only	1
(10)	Jack hammers	2
(11)	Concrete mixer 10/7	4

II. For Superstructure

(1)	Cencrete mixers	3
(2)	Welding set	1
(3)	Generating set	1
(4)	Compressor set	1
(5)	Steam winch	1
(6)	Trailer for macalloy bar	1
(7)	Grouting machine	1
(8)	Water pumps	2
(9)	Weigh batch double unit	1
(10)	Electric saw mill	1
(11)	Electric drilling machine	1
(12)	Electric grinder machine	1
(13)	Lee McCall 42 tons jacks	2
(14)	Hydraulic jack 60 tons 10 in. traverse	6
(15)	Buda Jack 12 in. capacity 20 tons traverse 12 in.	1
(16)	Lorries	3
(17)	Vibrators	
	Petrol	3
	Electric	4
(18)	Concrete cube testing machine	1
(19)	Trollies	2
(20)	Hand winches	3

PLATE I



KEY MAP SHOWING SITE OF MAHI BRIDGE

PLATE III

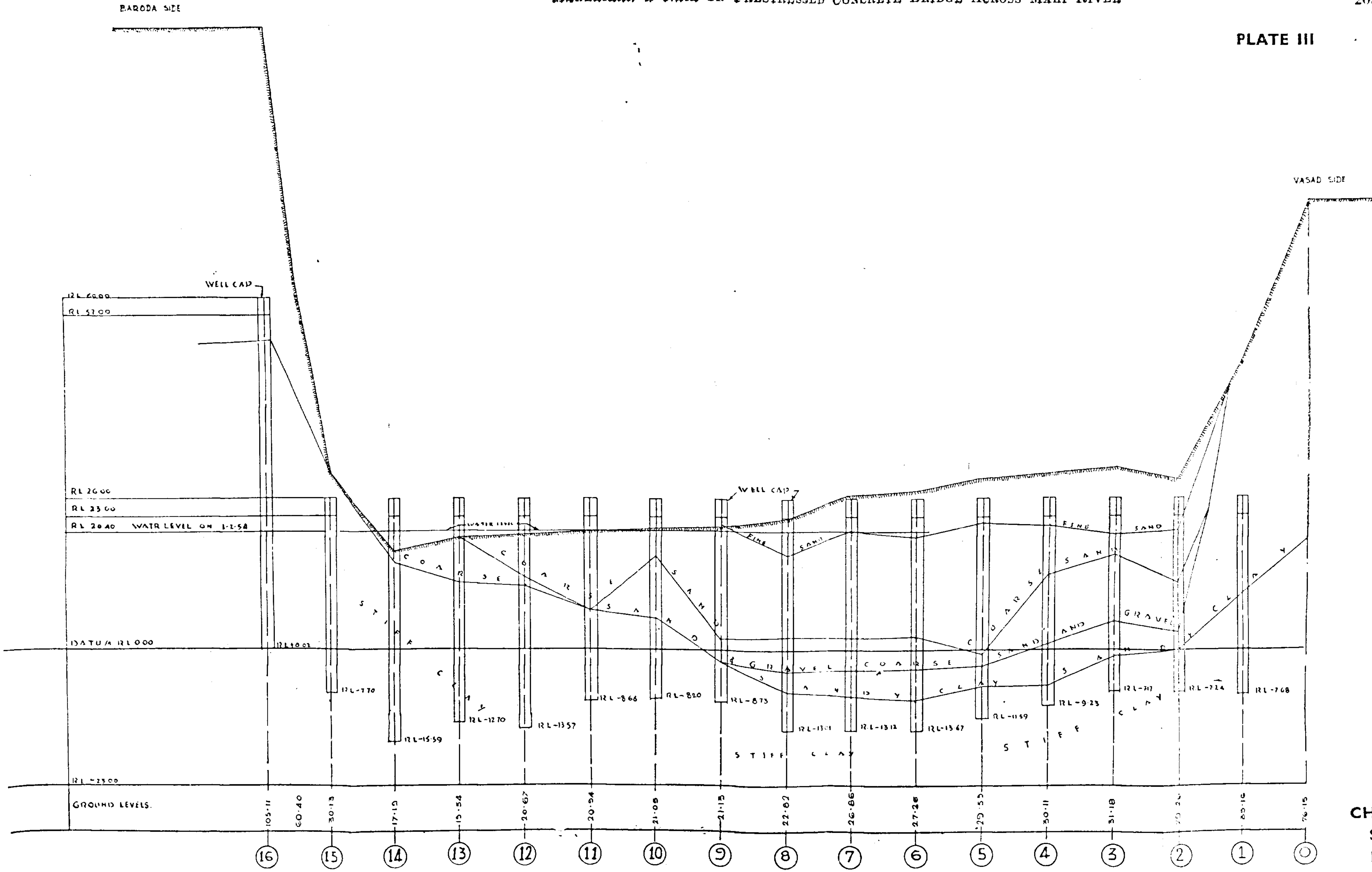
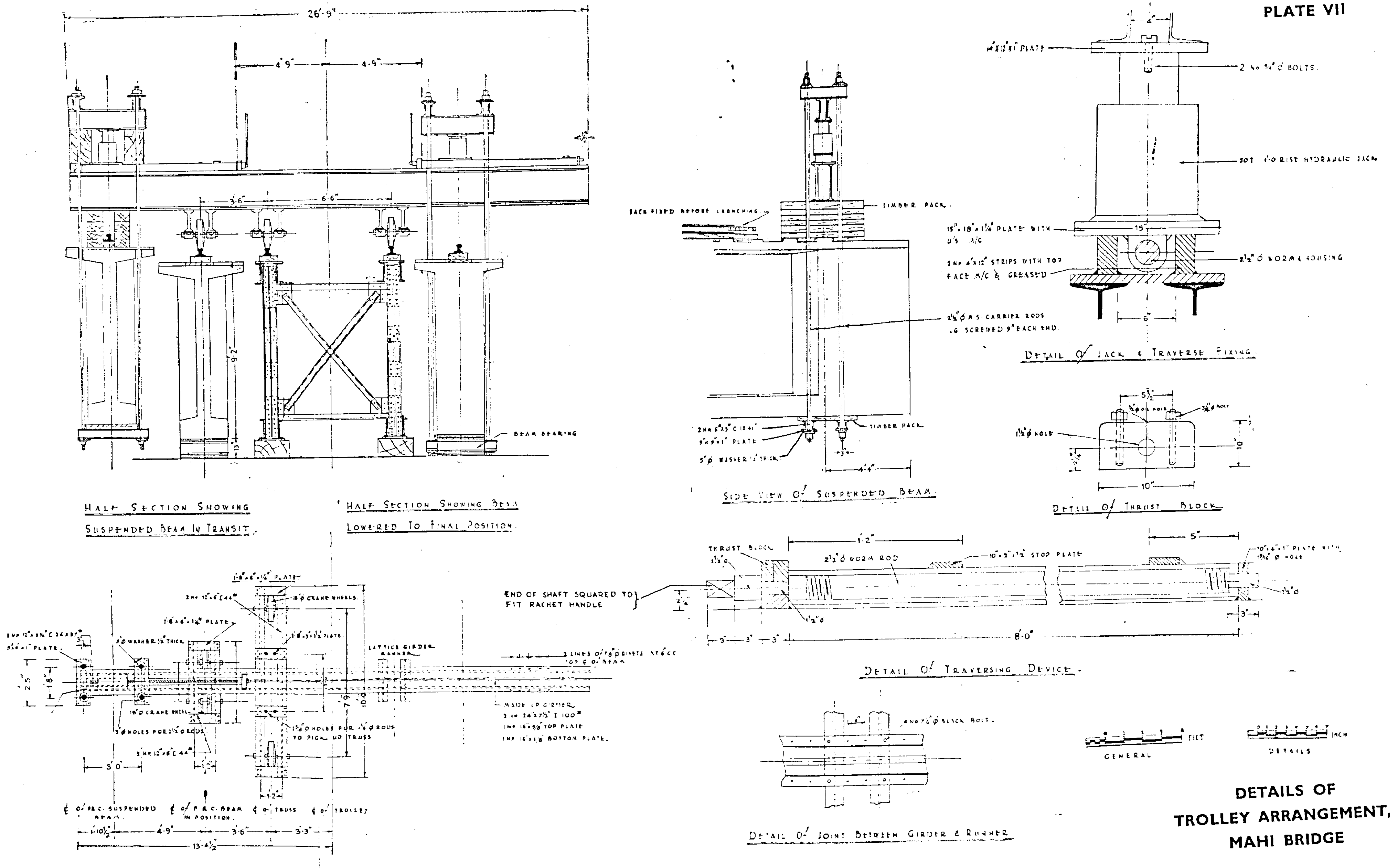


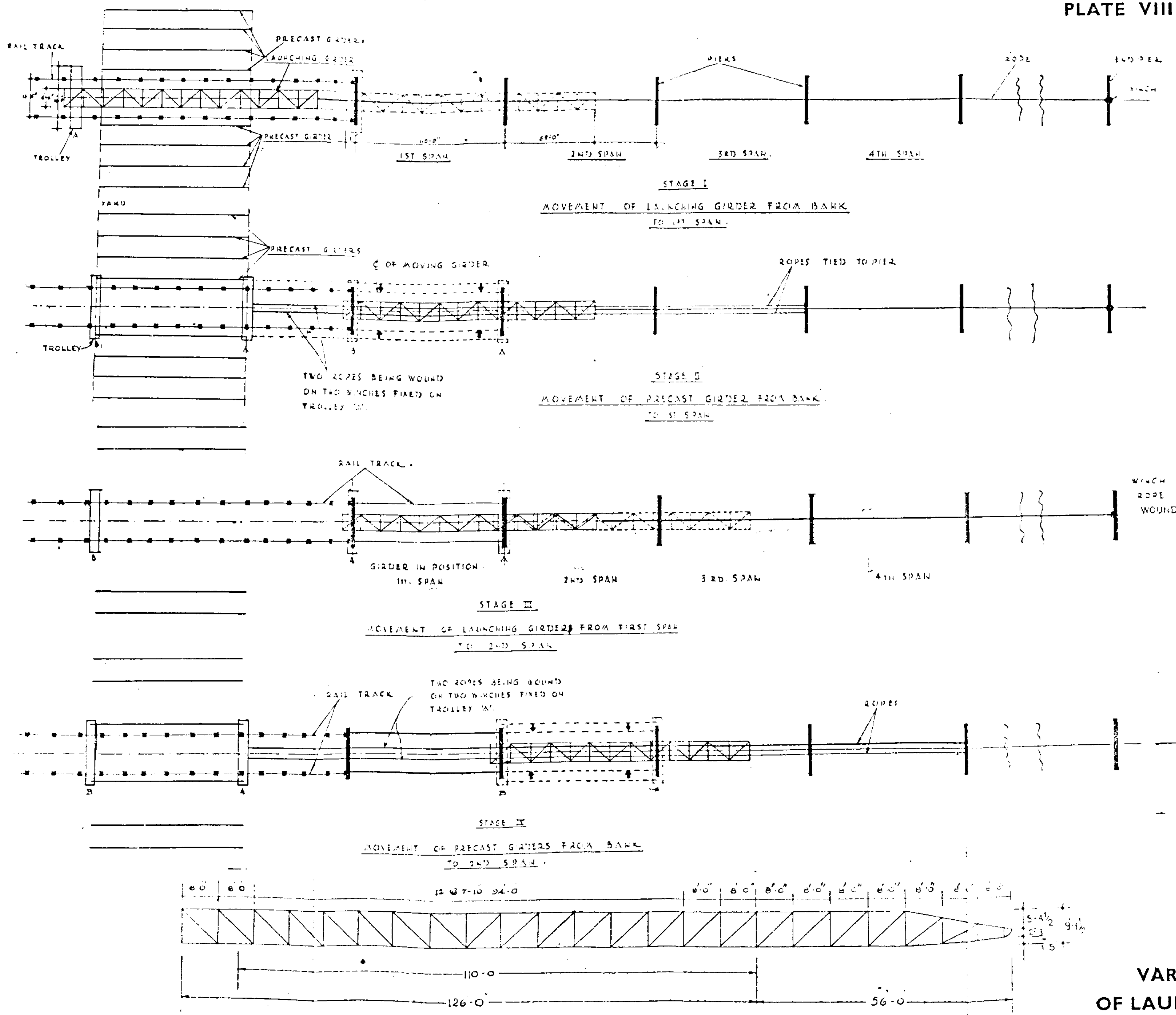
CHART SHOW
SOIL STRAT
MAHI BRIDC

PLATE VII



DETAILS OF TROLLEY ARRANGEMENT, MAHI BRIDGE

PLATE VIII



OUTLINE OF LAUNCHING GIRDER

VARIOUS STAGES
OF LAUNCHING GIRDERS,
MAHI BRIDGE

Paper No. 184

"SPECIFICATION FOR SURFACE DRESSING WATER-BOUND MACADAM ROADS" CRITICAL NOTES

By

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AND

P. N. Sen Gupta, B.C.E., M.I.H.E., A.M.I.E.

SYNOPSIS

Use of medium sand for blinding the first coat in surface dressing water-bound macadam roads is advocated. It is claimed that a good water-proof mat, which effectively seals the ingress of moisture into the macadam, is formed.

1. INTRODUCTION

1.1. A very large percentage of roads constructed in the second Five-Year Plan is likely to be of water-bound macadam surface dressed due to limited funds and low traffic intensity. For heavily trafficked roads on high banks it is not desirable to provide an expansive surfacing before complete settlement of the bank has taken place. In such cases surface dressing will protect the water-bound crust at a reasonable cost till the final settlement of the bank has taken place when a suitable type of surfacing can be safely constructed.

Also, where suitable equipment is not available, surface dressing offers the only solution to the problem of constructing a large milage of road within a short period of time. If machines like pressure distributors and gritters are available and there is large programme to provide continuous work for the machines. The speed of construction can be considerably increased. Where the maximum possible milage has to be modernised with limited funds, economy in cost is an important consideration for adopting such a specification.

Further, traffic conditions on most of the roads in India are such that no heavier specification than surface dressing is necessary. It is a waste of money to use heavier specification unless a large proportion of traffic using the road consists of bullock carts when a much heavier specification than surface dressing, may be more economical in the long run. Fortunately, it appears that with the improvement in the road system already achieved, lorry transport is fast replacing the bullock carts of the pre-war days, especially on long hauls. Further rubber tyred bullock carts are to be seen in increasing numbers although the rate of increase is not fast enough to indicate that they will entirely replace the iron tyred ones.

From the above it will be apparent that surface dressing will continue to be adopted for treating a large milage of roads.

1.2. It is the object of this Paper to establish that by a suitable adjustment of the specification based on an understanding of the various factors involved, a surface dressed road may give a much longer service than most of us normally think possible.

1.3. The function of surface dressing is to protect the water-bound macadam from deterioration under traffic. The water-bound macadam has to carry the loads imposed by the traffic and it should be adequately designed. Surface dressing will, however, not be able to correct an already corrugated water-bound macadam surface.

2. CAUSES OF DETERIORATION OF WATER-BOUND MACADAM

2.1. Although a water-bound macadam may have been properly designed to carry the expected traffic, deterioration is caused by the undernoted factors if it were untreated.

(i) Fast moving pnuemutic tyred traffic removing the binding material from the interstices of the stone metal by suction.

(ii) Tendency of the individual stone pieces constituting the water-bound macadam to get dislodged by the pounding action of the traffic.

(iii) Weakening of the foundation and the consequent sinking of the water-bound macadam and its ultimate disintegration by penetration of rain water into the subgrade.

(iv) Wearing of the surface due to abrasion by traffic.

2.2. Both the first and the second factors cause unravelling of the crust. Where there are efficient road maintenance gangs to replenish the blinding material as it is gradually displaced by fast traffic, predominantly uni-directional high speed traffic may cause corrugations on unsurface-dressed roads. The third factor causes rapid disintegration of the water-bound crust where subsidence has taken place. Generally the fourth factor hardly comes into play as the road would have been already destroyed by the first three factors if sufficient care has not been taken in construction and the maintenance of the road.

A suitable application of one or successive coats of surface dressing can protect the water-bound surface from all the deteriorating influences mentioned above and maintain its riding qualities indefinitely.

3. OBJECT OF SURFACE DRESSING

3.1. The primary object of the surface dressing is to prevent surface disintegration and weakening of the subgrade by stopping entry of water by the formation of waterproof mat. This can be achieved effectively only if the waterproof mat formed is uniform, has good bond with the blinding material and the stone metal through penetration of the binder into the former, and has no break which may allow rain water to penetrate into the subgrade.

Water entering through a break in the waterproof mat gradually spreads and in course of time helps in dislodging the mat from the water-bound macadam by decreasing adhesion. Thus a pothole is formed. If this pothole is not detected and immediately repaired, water enters in larger quantities and penetrates deeper, weakening the subgrade.

The surface dressing should, therefore, be applied with due care to ensure that no weak spot remains in the waterproof mat.

The weak spot may be caused by application of the binder on a dusty or an improperly cleaned surface, resulting in the binder film not being uniform or discontinuous. Or it may be caused by local penetration of most of the binder into a loosely compacted water-bound macadam leaving almost no binder to form a waterproof mat on the particular portion of the surface.

3.2. It should appear, therefore, that the primary object of the first coat of surface dressing is to form a mat which will be waterproof and satisfactorily bonded to the water-bound macadam over which it is applied. This can be achieved by:

(i) Selection of a suitable grade of road tar and determination of the correct quantity to be applied (depending on the texture of the water-bound macadam surface) so that after penetration sufficient quantity is available to form a waterproof mat in conjunction with the blinding material used.

(ii) Use of such blinding material as will enable sufficient quantity of road tar to stay on the surface and which will not cause the waterproof skin to break.

(iii) Use of such blinding material which will help in formation of a continuous film of road tar on the surface by spreading in case the pouring or spraying has not been able to form a continuous film owing to the water-bound surface being too dusty or loose.

As the texture of the water-bound macadam generally varies considerably, only experience can help determine how much road tar and of what grade should be specified in a particular case. The specification should permit a wide range for the first coat so that the exact quantity to suit the actual condition at the time of application can be sprayed.

4. MEDIUM SAND IS GOOD FOR BLINDING

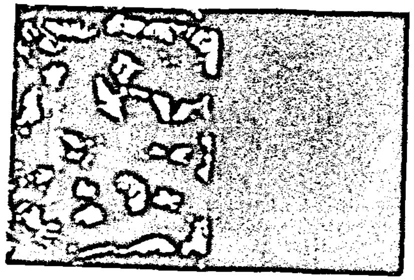
4.1. Use of medium sand as the blinding material for the first coat is desirable as it helps in forming a satisfactory waterproof mat by attracting sufficient quantity of road tar to the surface by capillary action. Further, it helps in making a non-continuous road tar film continuous by capillarity and the kneading action of the traffic when the sand becomes warm due to the sun.

4.2. Stone chippings on the other hand do not help in distributing the road tar so that any initial discontinuity in the tar film does not close up in most cases. Photos 1 to 3.

Further, the sharp corners of the stone chips sometimes cut through the binder film while being rolled and thus cause a break. This break hardly heals because the dusty blinding materials in the water-bound crust prevent it. Also, chippings falling over the stone metal of the water-bound macadam get crushed under the roller and cause a certain amount of discontinuity in the tar film.

4.3. Road tar has a tendency to penetrate into the water-bound crust. Hence the stone chippings cannot help keep enough of it on the surface to form the water-proof mat as they (the stone chippings) have no capillarity as the sand has. The first

Photo 1.



Blinded with chips. Blinded with sand.

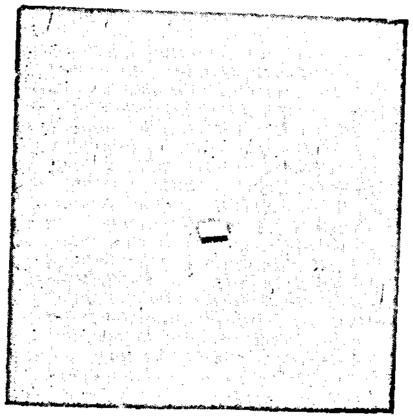


Photo 2. Surface blinded with sand. Photo taken 1 hour after rain.

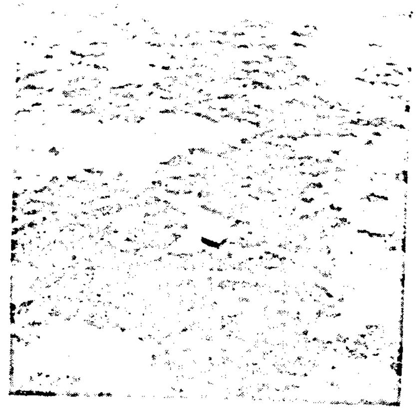


Photo 3. Surface blinded with chips. Photo taken 4 hours after rain.

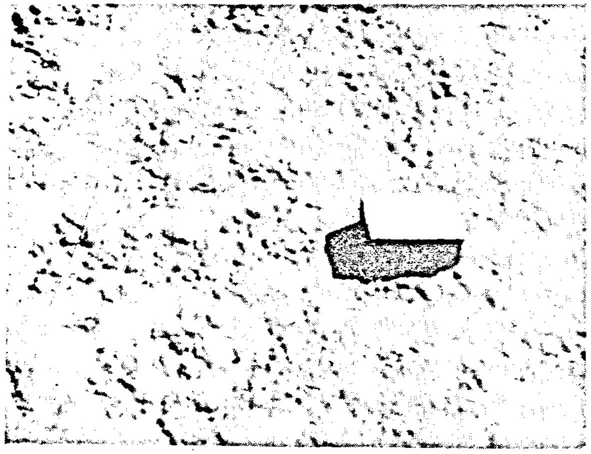


Photo 4. **Chippings getting dislodged due to moisture.**

Photo 5. **Two coats of surface dressing with stone chips blinding in both coats. Water has penetrated into the base.**

Photo 6. **1st coat blinded with sand & 2nd coat with chips. The base is completely dry after rain.**

coat surface dressing blinded with stone chippings will, therefore, remain rough and not be satisfactorily waterproof. Further, interstices between the chippings will hold water for a long time after rains and the moisture will soak in. Damp patches can be seen on such roads for many hours after a rain, indicating that the water has gone down deeper. No such patch will be seen on the surface blinded with sand. In case of sand blinding, the voids in the sand are completely filled up by road tar making the sand mat completely waterproof, Photos 5 and 6.

In addition moisture remaining on the surface for a long time will also cause displacement of the stone chippings by gradually reducing adhesion between the road tar and the stone chippings, Photo 4.

4.4. If stone chippings are used for blinding the first coat, those chippings that lie directly over the stone metal of the water-bound macadam will, under the impact of iron-tyred bullock cart wheels get crushed into several pieces which come off the surface as there is no binder at the lines of cleavage to hold them together. These broken particles lie loose on the surface and being pushed here and there by traffic action help in dislodging the chippings already firmly bedded in the road tar film. This defect can be rectified to a large extent if a wearing course blinded with chippings is laid over the sand mat. It has been found by experiment that the cushioning formed by the sand mat increases the load bearing capacity of the stone chippings of the wearing course to 2 to 3 times of what they could ordinarily bear when placed in direct contact with stone metal of the water-bound macadam. The reason for this may be due to :

(i) the larger supporting area for the chippings, and

(ii) the elastic nature of that supporting area.

Therefore, the most suitable specification for obtaining the best possible results would be as under.

5. SUGGESTED SPECIFICATION

5.1. First Coat

(a) Clean the water-bound surface thoroughly so that the surface of the individual metal is clean and the entire surface free from dust, so that road tar will bond firmly with the stone and the blinding materials. If the dust is so much that it cannot be properly removed, the surface may be very lightly sprinkled with water.

(b) Normally road tar grade 2 (if the surface is very porous and loose road tar grade 3) heated to a temperature of 220 to 240 deg. F. may be used. Apply by pouring or spraying. Brush out as evenly as possible. Blind with medium sand. Normally, 40 to 45 lb of road tar grade 2 and 3 cu. ft. of sand are required per 100 sq. ft Brush out the sand evenly and open to traffic. The sand has to be brushed back into position for the first few days as it gets displaced by traffic. If good quality sand is not available, stone grit may be used.

If the climate is very damp 45 to 50 lb of binder will be required per 100 sq. ft. In dry climate, the application may be reduced to 35 to 40 lb per 100 sq. ft.

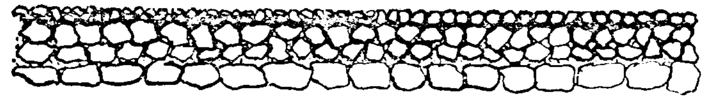
Overheating the road tar will reduce the life of the surfacing considerably by driving off the essential oils from it. The working temperature should be between 220 and 240 deg. F.

(c) When the tar works upto and flushes the surface making it uniformly smooth, apply the wearing course. Depending on the traffic and time of the year when the application has been given, it may take from 2 to 3 months for the surface to become smooth.

5.2. Second Coat

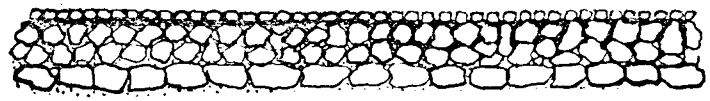
The wearing coat should be applied within six months of application of the first coat and in any case before the rains start. If it has to be applied during the rainy season, wet weather process is to be adopted.

As the surface has already become smooth all the road tar will help binding the chippings to the surface. 30 to 35 lb of road tar grade 3 per 100 sq. ft. blinded with 5 cu. ft. of $\frac{1}{2}$ in. stone chippings will be sufficient. It will be preferable to use single sized chippings instead of graded ones. When graded chippings are used, the smaller sizes will come into the way of larger chippings obtaining contact with the road tar at the time of application. In any case, a small size particle lying between the larger ones will have its surface lower down and will not help in bearing the traffic loads. It will lie loose as it will not be pressed by the roller wheels.



Surface blinded with graded chips

Surface blinded with one size chips



1st Coat blinded with sand & 2nd coat with chips

1st Coat blinded with chips

The application should be such that there are no loose chippings lying about on the surface. Chippings lying loose on the surface help in dislodging those adhering firmly by action of the fast traffic.

The chippings should be brushed out evenly to ensure that one particle does not lie over another. This will cause crushing of the chipping under roller wheels. The rolling should be continued till the chippings are bedded firmly on the surface but not crushed. A 6 to 8-ton roller should be used for this purpose.

It should be seen that the chippings do not disturb the underlying sand mat which will effectively stop access of rainwater into the subgrade.

5.3. Although the surface may look rough, yet it will dry quickly after the rain stops as the water will flow out passing around the chippings and without soaking into the surface.

An ideal surface would be one where road tar has flushed just up to surface of the chippings. This will ensure a waterproof as well as smooth riding surface without reducing its nonskid quality which is so essential to cater for increasing high speed traffic. The chippings should be hard and tough so that they do not get easily crushed under traffic. These should preferably be cubical and not flaky. Rounded gravel or river shingles may also be used with advantage provided it is hard and tough. Chipping or gravel or shingle must be clean and free from dust, otherwise adhesion with the road tar will be greatly impaired.

5.4. When a pothole becomes noticeable it must be repaired at once. A neglected pothole may cause serious damage to surfacing, especially if it appears during the rain, whereas the cost of repairing it is negligible. It is essential to cut out the pothole square and clean it thoroughly. Just the right quantity of road tar neither too much nor too little is to be applied. The painted pothole is to be blinded with stone chippings and rammed. If possible, precoated chippings should be used for blinding. The repaired surface of the pothole should be kept slightly proud of the adjoining surface so that it becomes level with the adjoining surface under traffic.

INFORMATION SECTION

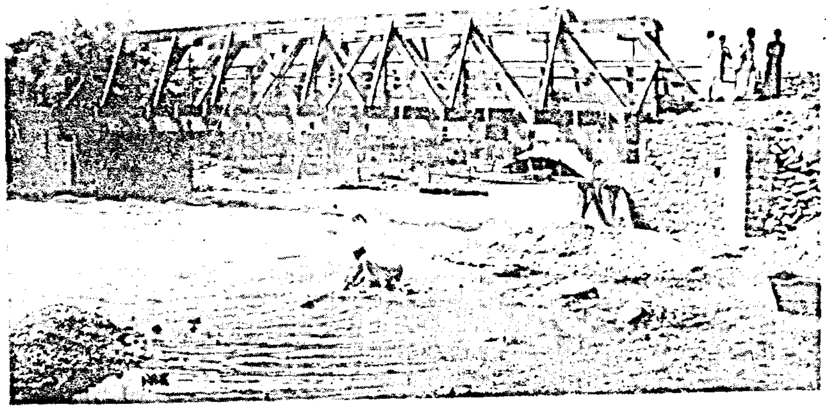


Photo 1. Howe Truss Bridge

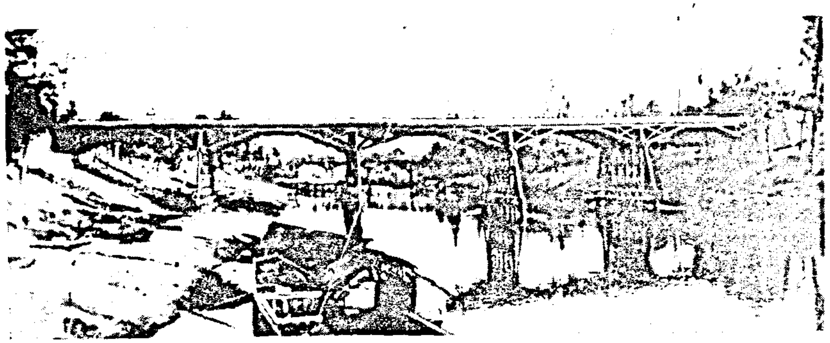


Photo 2. An understrutted Bridge

TIMBER BRIDGES IN KASHMIR

1. In the State of Jammu and Kashmir, which is mountainous, modern means of transport are lacking. Therefore cost of transport is heavy, and the engineers have mostly to rely upon the indigenous materials for construction work.

Various types of timber structures over bridle roads for light loading and of short spans have been in use in the State for a very long time. These have gradually been developed for heavier loading and longer span and, of late, most of the major bridges in the State have been built with timber.

The type mostly adopted is the under-strutted bridge, but Howe-truss and cantilever types are also adopted wherever these are more suited and economical for heavier loading.

2. CANTILEVER BRIDGES

2.1. Initially, bridges of this type were constructed up to 100 ft spans over bridle roads for pedestrian and light mule traffic. Later, their design was improved and cantilever bridges up to 75 ft span were constructed for heavier loading.

The wooden cantilever beams are laid to a minimum slope of 1 in 4 and are suitably anchored in masonry abutments which have to be sufficiently heavy to counterbalance both live and dead loads of the bridge. But the drawback of this type of bridge is that its members have to be of heavy sections which are difficult to procure locally in certain areas and their transport from distant forests is difficult and costly.

The main advantages are:

(1) For bridging torrential streams, flowing in deep gorges, no scaffolding and central support are necessary during construction.

(2) The method of construction is so simple that highly skilled labour is not necessary.

(3) The masonry abutments above the normal water level, which also serve as counterweights, are constructed in dry rubble with wooden cribs which is very economical and easy to construct. Besides, the anchored

portions of the cantilevers along with the cross bracings embedded in dry masonry have much longer life than if these were embedded in lime mortar masonry.

This type of structure becomes uneconomical for loadings heavier than class 12 and spans longer than 75 ft due to heavy sections and unmanageable lengths of the beams required.

Details of a contilever bridge, 74 ft span, are given in Plate I.

3. HOWE TRUSS BRIDGES

3.1. Howe truss bridges were originally constructed up to 50 ft spans for light loading but, of late, bridges of this type have been constructed up to 80 ft span and for Class 12 loading.

The principles of design of these bridges are the same as that of similar type of steel bridges. While the members in compression are of timber, the tension members are of steel rods. Loads through the compression members are transmitted at joints through steel plates. Hard wood chocks properly seasoned are used as gussets.

3.2. Construction of this type of bridge demands high class workmanship, close supervision and sound seasoned timber, as otherwise with the passage of time due to shrinkage of timber, the trusses get distorted. Generally, timber decking is provided. R. C. C. decking cannot be adopted because the chances of distortion of the trusses are still there.

3.3. Recently, a 132-ft long bridge of two spans of 66 ft each has been constructed over Sandra nullah on Anantnag-Verinag road. The cost of construction, including masonry works, comes to Rs 900 per linear foot.

Details of the truss are given in Plate II. This type of bridge is illustrated in Photo 1.

4. UNDER-STRUTTED BRIDGES

4.1. This type of construction is the most economical and widely adopted all over the State. It was originally introduced 40 years ago by Lt.-Col. Frazer, R.E., the then State Engineer, for spans up to 50 ft and for 6-ton roller load. Subsequently, with the development of technique and improvement

of design, spans up to 65 ft were constructed for 8-ton roller load. After 1947, bridges up to 85 ft span for single and double lanes of traffic for Class 18 loading have been constructed. The design of the under-strutted bridges for varying spans from 20 ft to 85 ft have been standardized for Class 12 and Class 18 loadings.

4.2. The outstanding features of this type of bridge are :

(i) The design is simple and easy to execute.

(ii) Owing to comparatively lighter sections and smaller lengths of the various members, this type of bridges can be constructed even in remote areas.

(iii) The seasoning of timber is not essential.

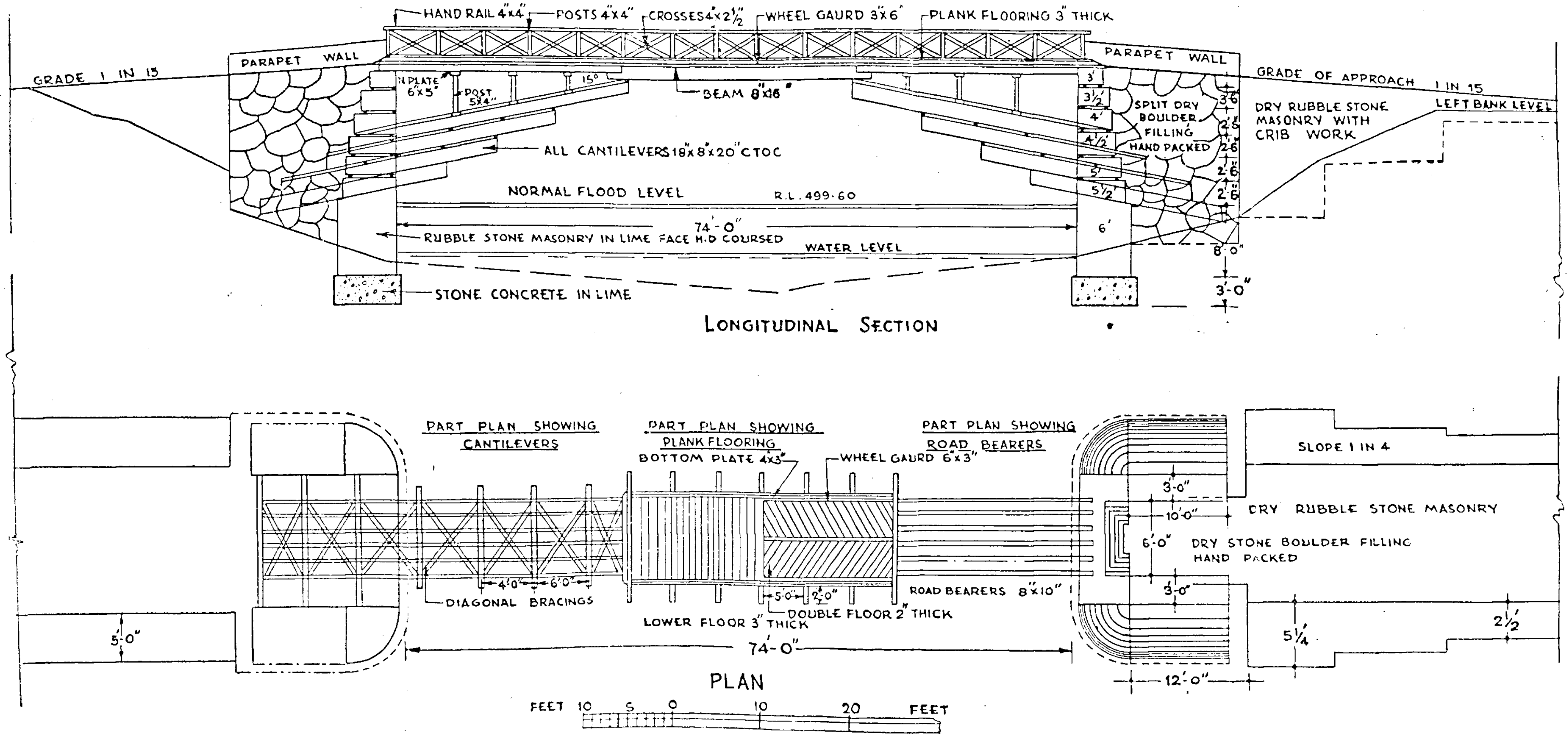
(iv) The cost of construction of this type of structure in the State is comparatively low.

4.3. In this type of structure, the road bearers and the straining beams are designed as simply supported. Normally, the length of the beams does not exceed 20 feet. The camber of 1 in 50 to 1 in 60 has been found suitable to allow for subsequent sagging due to shrinkage of timber.

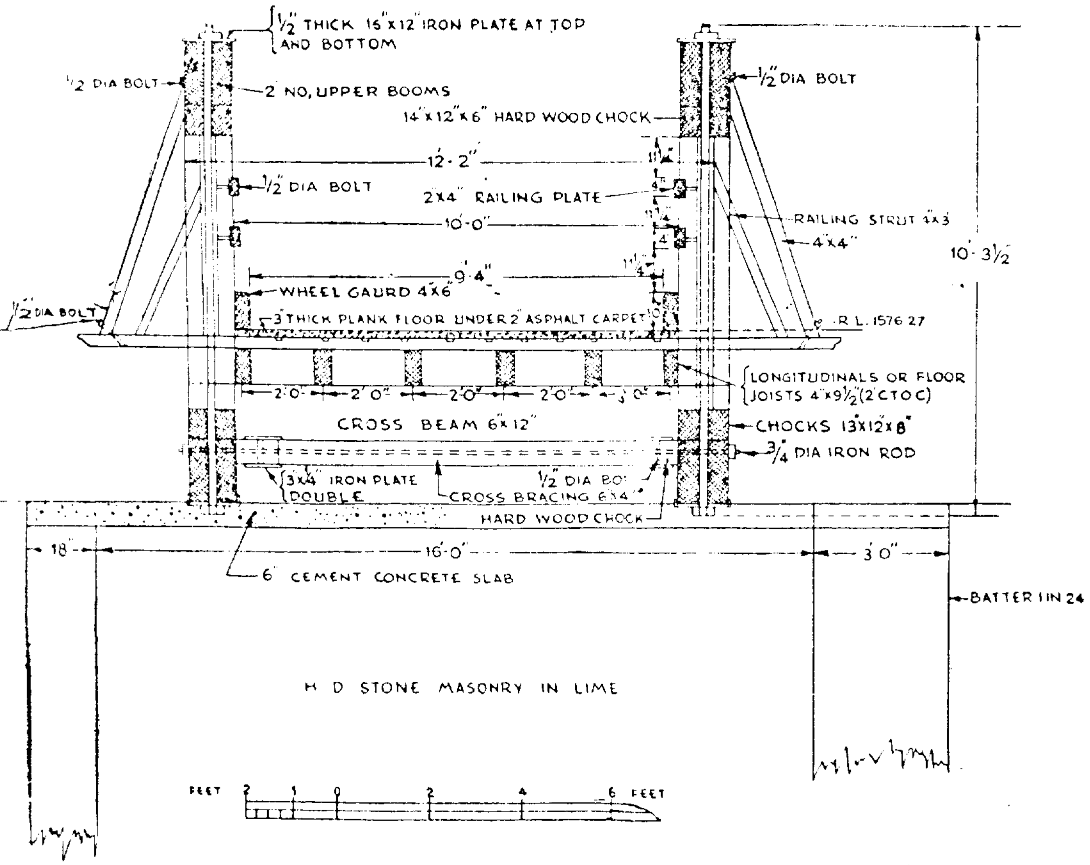
Photo 2 shows a bridge, which is 257 ft long, of 5 spans of 55 ft each with a carriageway of 22 ft and 6 ft wide side walks on either side designed for the old I. R. C. Standard light loading. The cost of this structure is 2.25 lakh which works out to Rs 820 per linear foot.

Plate III gives details of an under-strutted bridge of 50 ft span.

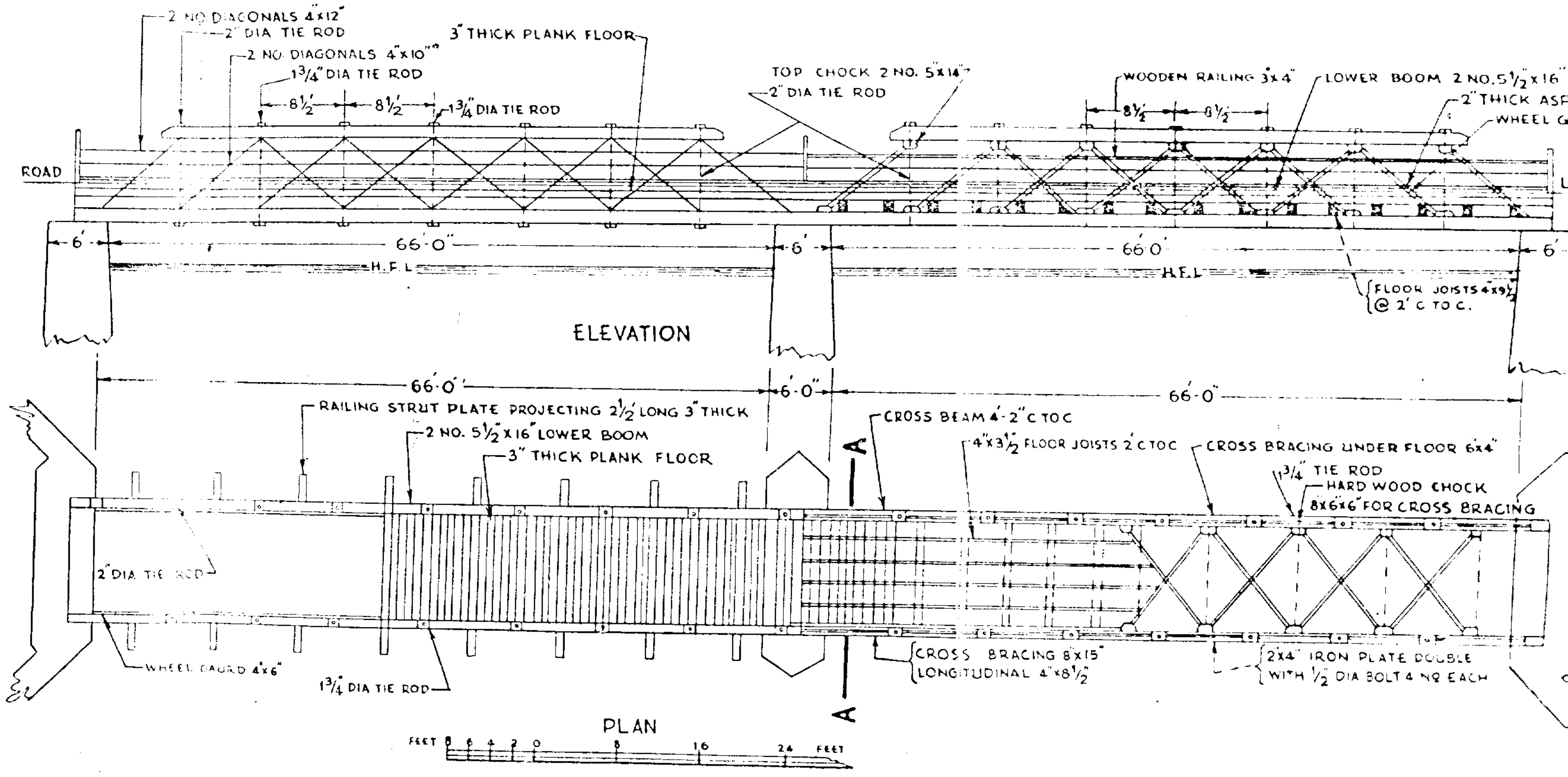
PLATE I

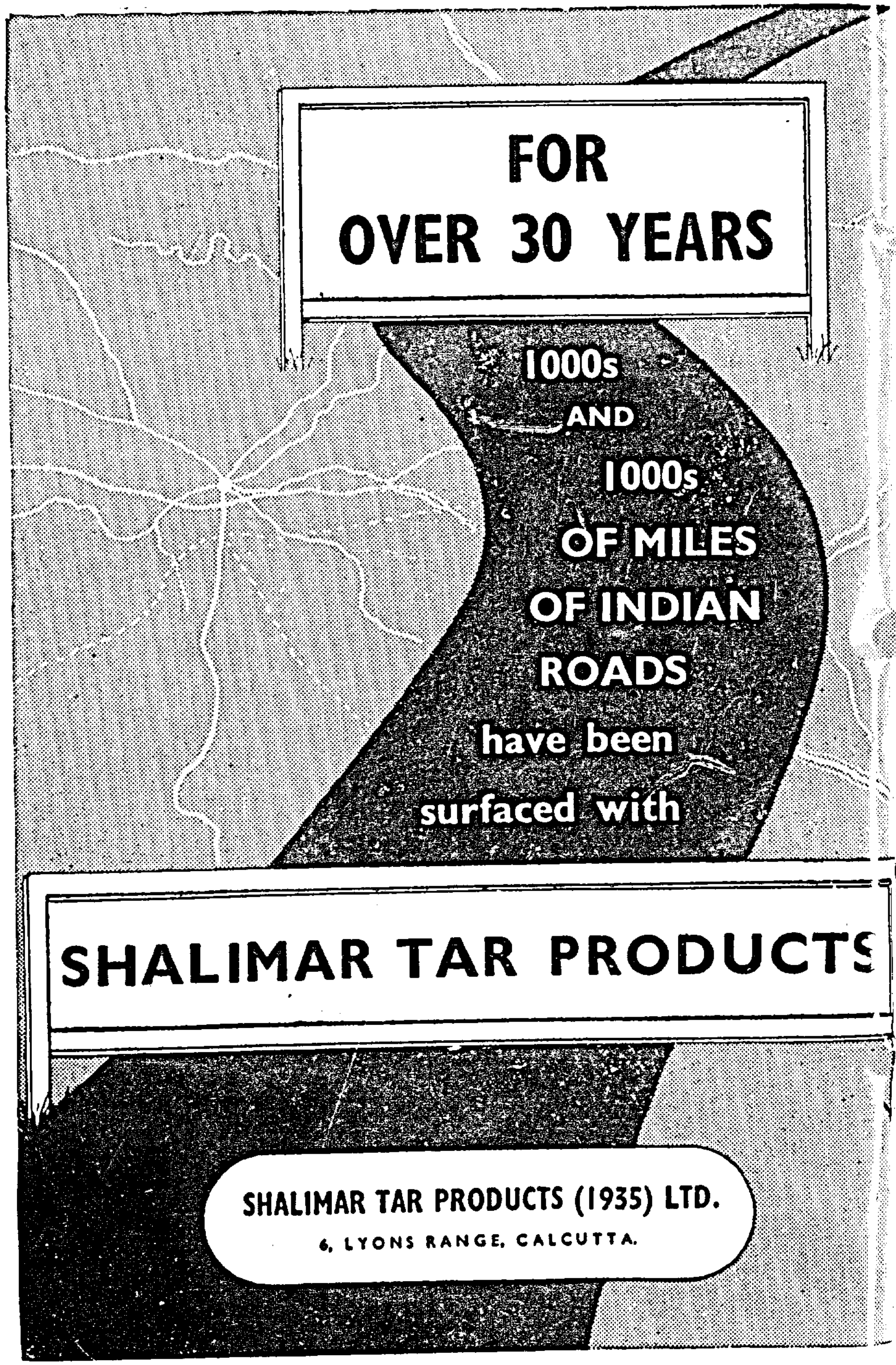


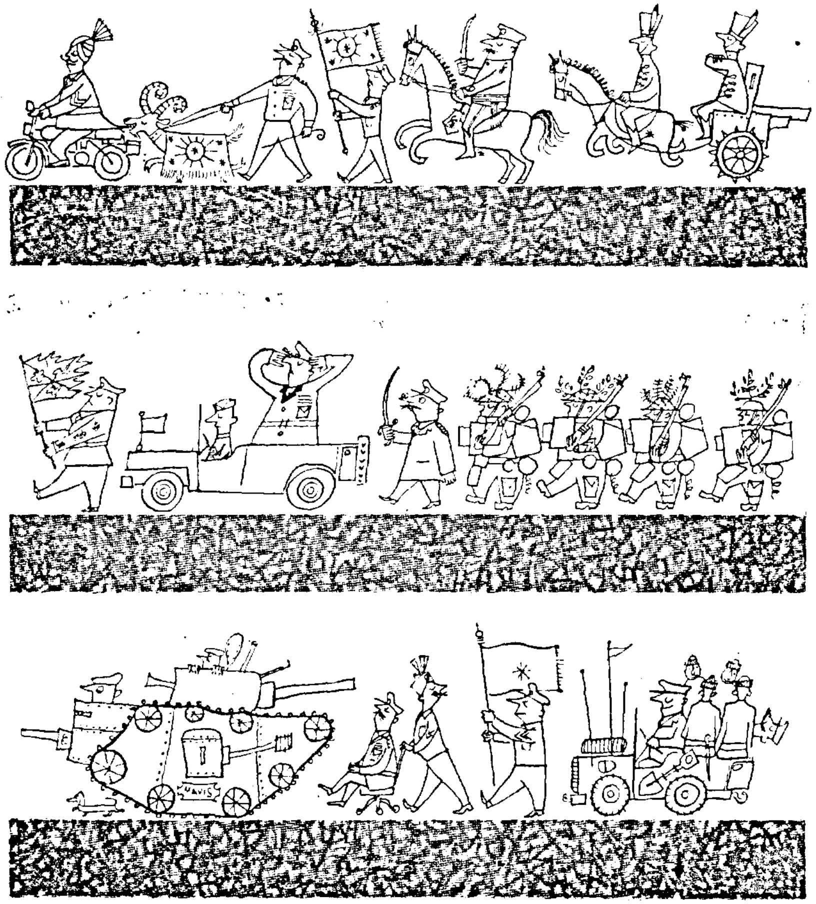
TIMBER CANTILEVER BRIDGE
SPAN 74 FEET

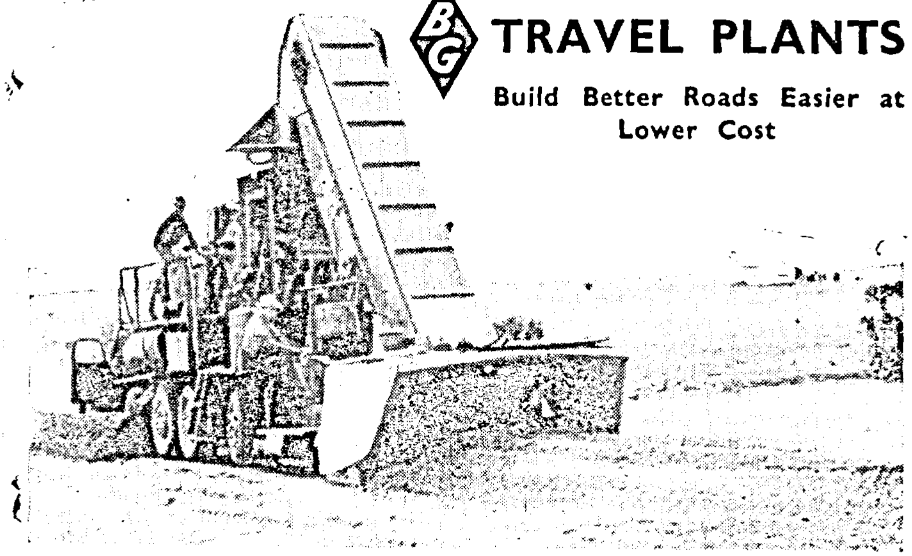


CROSS SECTION ON A A




FOR
OVER 30 YEARS
1000s
AND
1000s
OF MILES
OF INDIAN
ROADS
have been
surfaced with
SHALIMAR TAR PRODUCTS
SHALIMAR TAR PRODUCTS (1935) LTD.
6, LYONS RANGE, CALCUTTA.

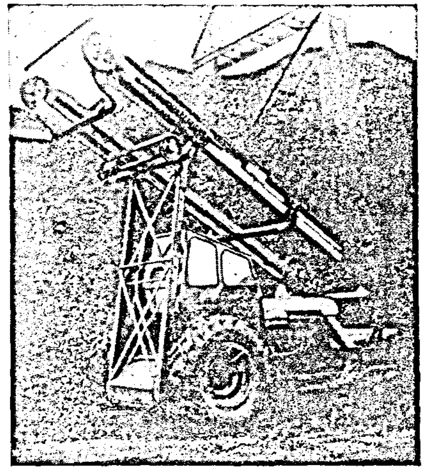




Reprinted from HIGHWAYS AND BRIDGES AND ENGINEERING WORKS, *November 24th, 1954.*

New Features for the Merton Overloader

After two years of success in Britain and in the 22 countries to which it has been exported the British made Merton Overloader now has several important new features. As a result of this success Merton Overloaders are now being exported to the United States of America, Canada and parts of South America. The sole concessionaires in the United Kingdom and in the Americas are Mackay Industrial Equipment Ltd., of Faggs Road, Feltham, Middlesex, who tell us that this is the only British construction equipment now being exported to the U. S. A.



The improved Merton Overloader—the only British construction plant being exported to the U.S.A.

The bucket of the new Merton Overloader, Mark V, has a capacity of ¾ cu. yd. and covers the full width of the front wheels of the tractor. This eliminates any tendency for the front wheels to ride up on the material being loaded and also greatly increases the efficiency of the machine when excavating.

The digging depth of the bucket can be adjusted, simply by the removal and replacement of two bolts, to any position between 2 in. above ground level and 4 in. below ground level. The adjustment to 2 in. above the horizontal will simplify the operation of the machine when it is loading a stockpile on uneven ground when it is important that water or foreign matter from the base should not be mixed with the material being loaded.

The discharge height of the Merton Overloader is now variable. A small hand winch, operated by one man, raises and lowers the overhead structure of the machine by extending and retracting the telescopic rear upright members so that the bucket can be discharged at any height from a minimum of 6 ft. 11 in. to a maximum of 11 ft. 2 in. To change from minimum to maximum discharge heights takes only three minutes and entails the removal and replacement of only four bolts. The reach of the bucket, that is the distance from the inside edge of the bucket when in the discharge position to the rearmost part of the tractor (the tyres), is 3 ft. 6 in. when in the minimum discharge position and 2 ft. when in the maximum discharge position.

From the minimum discharge position the overhead structure can be lowered still further to the travelling position giving the machine an overall height of 8 ft. 5 in.

An enclosed cab is now a standard fitment on the Merton Overloader. It is constructed of heavy gauge sheet steel and fitted with toughened safety glass windows, which give the operator excellent all round vision. The turning radius of the Mark V Merton Overloader is the same as that of the Mark IV—17 ft. 6 in.

A light materials bucket of 1 cu. yd. capacity is available as an optional extra. Other optional equipments are a 6 ft. 9 in. bulldozer blade and a one ton capacity crane hook attachment.

The Merton Overloader of course, does not turn or slew, it shuttles backwards and forwards in a straight line between stockpile and lorry. The power unit is the Fordson Major diesel engine.

The complete cycle from loading the bucket to discharging and back to loading takes 15–20 sec., and during this time the machine can also travel backwards and forwards a distance of twelve yards. It is claimed that it can load blasted rock at the quarry face at the rate of one ton per minute or sand from a stockpile at the rate of 5 cu. yd. in 2¼ minutes.

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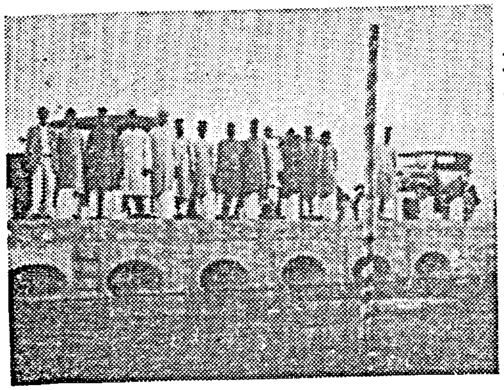
For Water Supply, Drainage, Culverts, Dust Bins, Cable Conduits, Septic Tanks, Channels, Tanks, Sentry Cabins, Cattle Troughs, Latrines, etc.



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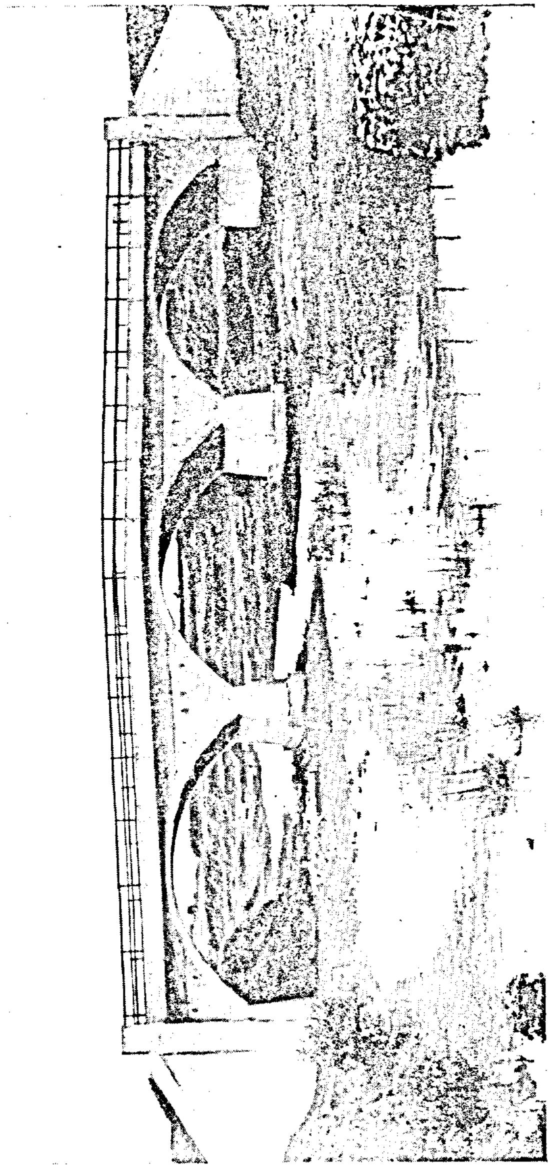
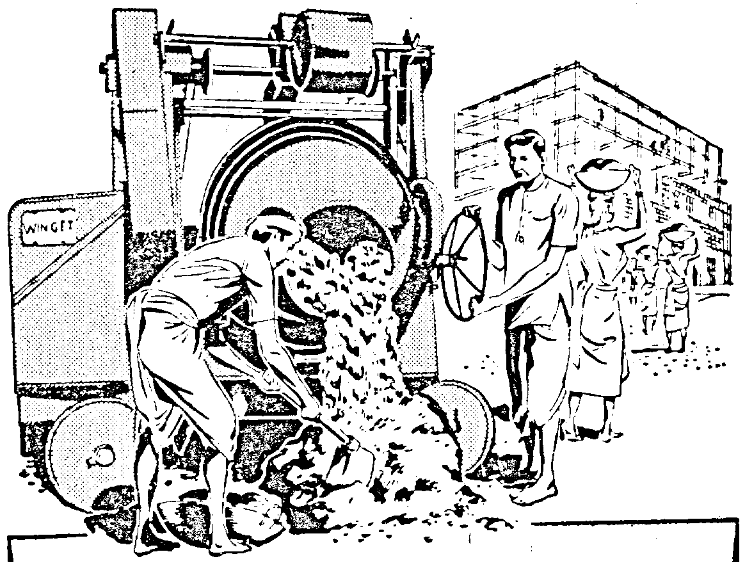
We request the pleasure of your visit to Walchand Group Stall No. B-37 at THE INDIAN INDUSTRIES FAIR, DELHI, 29th October to 15th December 1955.



LANJEE BRIDGE.—This is a small plain concrete barrel arch bridge with three spans of 33 ft. each between centres of piers. It carries a clear roadway of 20 ft. and the foundations have been taken down by open excavation into the sandstone, 10 ft. below low water level. Designed and constructed by Gammon India Ltd. on the Kharsia to Ambikapur Road to the orders of the Chief Engineer, Madhya Pradesh, P.W.D. India.

GAMMON INDIA LIMITED

HAMILTON HOUSE BALLARD ESTATE BOMBAY-1


WINGET