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Paper No. 219

# " SCIENTIFIC DESIGN AND LAYING OF A BITUMINOUS CARPET ON A NATIONAL HIGHWAY "

*By*

S. R. Mehra*, C.G. Swaminathan * and V.R. Vaish†

**CONTENTS**



**SYNOPSIS**


The Paper deals with the various aspects of providing an asphalt carpet commencing from the stage of scientific design to actual execution of the work on the Delhi-Mathura Road, National Highway No. 2. The Consulting Engineer (Road Development), at the instance of the Director, C.R.R.I. agreed, as the financing authority, to the project being treated on a proper scientific level, not only in design but also in execution.

Based on CBR test, the thickness of the road structure to carry the estimated intensity of traffic was arrived at. Then a survey of local quarries and crushers was made. Based on an analysis of the materials available and keeping an eye on cost, a design for the asphalt mixture was worked out after conducting a series of tests in the laboratory.

The continuous mixing plant which was used for the construction was also calibrated so as to give the required mixture, and the various phases in the plant control and operation are discussed. The carpet was laid by a mechanical paver and finisher. Tests on the mixture before and after laying were carried out periodically.

The importance of scientific design, the role of field control laboratory, the details of mechanised construction and the test structure are all dealt with in the Paper.


* Central Road Research Institute, New Delhi.

† Central P.W.D., New Delhi.

## 1. INTRODUCTION

1.1. In order to cater to the ever-increasing demands of heavy traffic on the Delhi-Mathura Road, it was proposed to reconstruct it as well as the by-passes in the Delhi State, to start with, so as to have a dual carriageway of 22 ft width running parallel to the 22 ft wide old road which forms one of the two carriageways. It is obvious that whenever a highway reconstruction is envisaged, two of the important things which have to be decided are: the overall thickness of the pavement structure required to meet the traffic requirements, and the constitution of the various layers of the pavement.

1.2. On a traffic survey on the old road, it was observed Table 1, that the intensity of traffic was of the order of 4,000 tons per day and, as such, it can be classified as heavy particularly because the traffic is increasing. With a view to see as to

TABLE 1

**Traffic intensity on the Delhi-Mathura Road for 24 hours**

| | | |
|---|---|---|
| Motor traffic weighing over 3 tons | = | 3360 tons |
| Motor traffic weighing less than 3 tons | = | 1004 tons |
| Animal drawn vehicles, wooden tyred | = | 48 tons |
| Animal drawn vehicles, iron tyred | = | 96 tons |
| Animal drawn vehicles, rubber tyred | = | 192 tons |
| | | 4700 tons |

whether the existing road would require additional strengthening or not, to meet the demands of the heavy traffic, a series of field tests were conducted on the road bed. Based on the CBR data it was noted that the overall thickness of the crust required for the given traffic intensity should be 15 in. comprising at least 9 in. of soling with 6 inches of water-bound macadam over a well compacted natural subgrade. The existing road had a thickness of 11 to 12 in. and the second carriageway had also been constructed to the same thickness. Hence it was suggested that an additional layer of water-bound macadam of about 3-3½ in. of compacted thickness should be superimposed over them.



1.3. In order to have a smooth riding surface, it was also proposed to adopt a bituminous carpet of 1-1½ in. with a minimum thickness of 1 in. to be laid over the water-bound macadam. Since a dense carpet would mean a heavy expenditure, a directive was given to design a mixture which would not be open graded but as dense as possible, using the locally available aggregate and keeping in view the limitations of the construction plant proposed to be employed and other operational problems to be confronted.

1.4. This Paper is an outcome of the joint efforts of the Central Road Research Institute and the Delhi State P.W.D., towards a scientific handling of a bituminous carpet work and serves as a typical example of the teamwork between a research organisation and an executive department, resulting in better work and greater economy. An analysis of the quantities required and the cost structure are given in Appendices I and II.

## 2. COMPARISON OF BITUMINOUS MIXTURES

2.1. The usual and the most common carpeting practice in this country is to lay an open-graded premix carpet consisting of about 10 cu. ft. of ½ in. stone per 100 sq. ft. of surface, mixed with hot bitumen at 3½ lb per cu. ft. of stone, and to compact the mixture to a thickness of about 1 in. To waterproof this, a seal coat is given consisting of about 3 cu. ft of ¼ in. grit mixed with 20 lb of bitumen per 100 sq. ft. of surface. The structure is usually quite open and there is a high percentage of voids in the mixture. To reduce the voidage and to give additional strength, in certain specifications a "crete" type of mixture consisting of about 8 cu. ft. of ¾ in.-¼ in. stone and 4 cu. ft. of coarse sand mixed with hot bitumen at 4 lb per cu. ft. of stone and 8 lb per cu. ft. of sand is laid per every 100 sq. ft. and compacted to give a thickness of about 1 in. It is obvious that neither of the two can give a strong and voidless mat. For, whereas in the open graded carpet there are no smaller sized chips to give sufficient interlock and to reduce the voids, in the latter case, there is no definite and clear gradation specified either for the stone chippings or for the coarse sand. The term "coarse sand" itself is vague and a wide variety of sands is likely to be employed and it is most unlikely that every time the final mixture would yield a strong and sealed mat. It is but necessary, therefore, that the materials should be so proportioned that sufficient interlocking of the aggregates is obtained. This proportion should take into consideration the quantity, different sizes of stone chippings as well as the particle size gradation of the sand used together with the optimum quantity of binder to be employed, for which a detailed laboratory design should be worked out.

2.2. Table 2 gives a comparison of the properties of the three different types of mixtures, namely : (a) designed mixture, (b) stone-sand 'crete' type of mixture, (c) open graded premix. From the point of view of voids it can be seen that a proper

TABLE 2

**Comparison of the properties of the mixture**

| No. | Properties | Designed mixture (Appendix II) | Stone-sand crete (Appendix IV) | Open graded premix (Appendix III) |
|---|---|---|---|---|
| 1. | Percentage of voids | 7.4 | 9.06 | 25 |
| 2. | Stability (Marshall) | 1286 | 9.65 | (sample fails immediately after moulding) |
| 3. | Flow | 15 | 14 | |
| 4. | Abrasion per cent | 0.48 | 0.54 | |

proportioning of the materials can yield a tight mat with as low a voidage as 7 per cent whereas the crete has a voidage of 15 per cent and the open graded contains air voids as high as 25 per cent. Again, the strength values indicate that a designed mixture has a very high inherent stability as compared to the other two. So it can be appreciated that, by proper proportioning of the aggregates and by adopting scientific design procedures, a strong and stable carpet can be obtained.

2.3. Table 3 gives the unevenness, measured with a bump integrator, of various types of bituminous surfaces. From the Table it is very clearly seen that the unevenness for the designed carpet when laid with a mechanical paver is the lowest and compares as favourably as a cement concrete slab. It is but obvious that such a surface would provide a high degree of riding comfort and enable a free and smooth flow of traffic.

2.4. One may say that such a designed mixture will be extremely costly. In Appendices II, III and IV are given the cost analysis for the three types of mixtures, namely, (a) designed



TABLE 3

**Bump integrator data for unevenness for different surfaces**

| | Name of surface | Index | Remarks |
|---|---|---|---|
| 1. | Water-bound macadam freshly constructed | 220 | |
| 2. | Surface dressing single coat | 262 | 1 to 6—surfaces on the same stretch of the road. |
| 3. | Surface dressing 2 coat | 258 | |
| 4. | Premix open graded | 250 | |
| 5. | Bituminous grouted macadam | 324 | |
| 6. | Designed carpet | 107 | |
| 7. | Premix with seal coat | 160 | |
| 8. | Cement concrete | 119 | |

dense mixture (b) the open graded carpet with a seal coat, and (c) the crete type of mixture. The cost for 1 in. thick carpet for 100 sq. ft. of surface for all the mixtures when laid with mechanical plant is Rs 24 only. Furthermore, the maintenance cost for the designed carpet will be very much less, since a separate seal coat, if at all necessary, can be deferred to a much later date than in the case of an open-graded carpet.

2.5. Therefore, considering the strength and stability of the proportioned mixture, taking into account the riding comfort of the finished carpet and, above all, from the point of view of ultimate economy in the cost, one cannot over stress the vital role that a design laboratory can play and the paramount importance of scientific procedures of design and construction.

## 3. DESIGN OF THE BITUMINOUS CONCRETE MIX

3.1. Normally, a graded mix should be designed in such a way that the percentage of voids in the compacted mix is limited to 5 to 10 per cent and also such that it has adequate stability. To achieve this, the aggregates used for the mix should be graded so as to conform to some standard specification or other, vide Tables 4 and 9.

TABLE 4

**Criteria for satisfactory design of a bituminous paving mixture**

| Properties | Marshalls Method | Hveem's Method | Asphalt Institute |
|---|---|---|---|
| Stability | Min. 500 lb for medium | 30 + for light | — |
| | Min. 1000 lb for heavy | 35 + for heavy | — |
| Flow or cohesiometer | Max. 20 | 50 + | — |
| Percentage voids in total mix | 3–5 | Min. 4 | 5-10 |
| Percentage voids filled with bitumen | 75–85 | | |

TABLE 5

**Physical properties of the quartzite aggregate**

| | |
|---|---|
| Crushing value (B.S.) | 23.68 per cent |
| Los Angeles Abrasion loss | 25.80 per cent |
| Impact value | 26.40 per cent |
| Specific gravity | 2.66 |
| Absorption (water) | 0.20 per cent |
| $SiO_2$ | 97.00 per cent |

In this connection, one may be justified in questioning as to whether it is always necessary to adopt completely the practices of the western countries. No doubt it is true that in order to obtain as maximum a density as possible for the mineral aggregate combination, the gradation should be chosen so as to be near the Fuller's curve and so as to fall within the limits set forth by foreign organisations such as the Asphalt Institute. But then, it is equally possible to have a gradation slightly outside the limits, and to obtain a mixture which would satisfy the strength and stability requirements. Again, it is not always necessary



TABLE 6

**Sieve analysis of the ½ in., ⅜ in. and crusher run materials collected from two crushers and storage bin of mixing plant**

| Sieve size or No. | Percentage by weight retained on each of the sieves used | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Crusher 1 | | | Crusher 2 | | | Storage bin | | | Average | | |
| | 1/2 in. | 3/8 in. | C. dust | 1/2 in. | 3/8 in. | C. dust | 1/2 in. | 3/8 in. | C. dust | 1/2 in. | 3/8 in. | C. dust |
| 3/4 in. | 0.8 | — | — | 1.8 | — | — | 1.2 | — | — | 1.3 | — | — |
| 1/2 in. | 60.3 | 1.2 | — | 65.1 | 2.8 | — | 67.6 | 2.2 | — | 64.3 | 2.1 | — |
| No. 4 | 37.6 | 69.7 | 0.3 | 32.5 | 48.5 | 0.1 | 31.2 | 52.9 | 0.3 | 33.8 | 57.0 | 0.2 |
| No. 10 | 0.7 | 24.3 | 1.4 | 0.3 | 38.1 | 1.6 | — | 29.1 | 3.1 | 0.3 | 30.5 | 2.0 |
| No. 40 | 0.6 | 4.3 | 49.6 | 0.3 | 9.0 | 51.5 | — | 14.3 | 50.9 | 0.3 | 9.2 | 50.7 |
| No. 80 | — | 0.3 | 28.3 | — | 0.8 | 26.4 | — | 0.9 | 28.7 | — | 0.7 | 27.8 |
| No. 200 | — | 0.1 | 12.6 | — | 0.4 | 11.3 | — | 0.4 | 11.8 | — | 0.3 | 11.9 |
| Passing No. 200 | | 0.1 | 7.8 | | 0.4 | 9.1 | | 0.2 | 5.2 | | 0.2 | 7.4 |

TABLE 7

**Specific gravity and unit weight for aggregate materials and the mixture (dry)**

| Material | Specific gravity (apparent) | Unit weight lb per cu. ft. | |
|---|---|---|---|
| | | loose | compacted |
| 1/2 in. | — | 82.7 | 95.5 |
| 3/8 in. | — | 89.4 | 102.5 |
| Crusher run | — | 87.5 | 103.1 |
| Mixture 2 : 3 : 2 (by vol.) ½ in. : 3/8 in. : C. dust (dry) | — | 105.5 | 118.8 |
| 25 : 45 : 30 by weight (dry) | — | 102.8 | 118.0 |
| Coarse aggregates 3/4 in.-No. 10 sieve | 2.657 | — | — |
| Fine aggregates No. 10-No. 200 sieve ... | 2.656 | — | — |
| Filler passing No. 200 sieve | 2.698 | — | — |

to have such a low air void as 5 per cent. A slightly higher air void in the mixture may be an advantage in the sense that there is room in the carpet to get compacted further without distortion as a result of traffic. In such a carpet, bleeding as a result of complete reduction of the voids due to subsequent traffic will not pose the problem of shoving and slipperiness at a later date. With this in the background it was attempted to get as dense a mix as possible possessing adequate strength and using the locally available aggregates to the best economic advantage.

3.2. The aggregate available is essentially a quartzite and its physical properties are as given in Table 5. The quarries from where the aggregates would be obtained were then inspected and it was found that the crushing plants usually give three sizes of materials, namely, ½ in., 3/8 in. and crusher run grit. Since the maximum size of the aggregates used should not be more than half nor less than one third of the compacted thickness of the carpet[3] and in as much as the minimum thickness of the carpet was fixed at 1-1½ in., it was decided that the maximum size of the aggregate used should pass ¾ in. with a slight tolerance for the over sizes. Further, since the crusher run grit was not in



TABLE 8

**Test properties of the mixture 25 : 45 : 30 by weight (½ in. : 3/8 in. : c. run) Bitumen 80/100 per cent by weight of total aggregates**

| Bitumen | 4.5 | 5.0 | 5.5 | 6.0 | Remarks |
|---|---|---|---|---|---|
| Marshall stability 938 lb (50 blows compaction) | 938 | 1150 | 1286 | 1096 | Lab. mix: Using average gradation of ½ in., 3/8 in. and cr. dust as in Table 3 for the mixture |
| Flow value 1/100 in. | 18 | 19 | 15 | 18 | |
| Per cent voids | 11.4 | 9.8 | 7.4 | 6.1 | |
| Unit weight lb per cu. ft. (compacted) | 137.4 | 139.0 | 142.8 | 142.7 | |
| Hveem stability per cent (compaction, double plunger 3000 psi) | 32.5 | 34.5 | 36.0 | 34.0 | |
| Per cent voids | 11.7 | 10.9 | 9.4 | 8.2 | |
| Unit weight lb per cu. ft. compacted | 136.9 | 137.3 | 138.6 | 139.5 | |

very great demand for construction purposes it could be obtained at a reasonably low price and is very much cheaper than the local sand. Gradation analysis of the three aggregates obtained from two quarries and the storage bin at the plant was done in the laboratory and is given in Table 6.

3.3. Although it may be possible to design a dense mix using these three materials, one has to consider the limitations that are enforced by the mixing plant. The mixing plant is essentially a continuous mix plant having a capacity of 40 tons per hour, Fig. 1 and Photos 1 and 2. The coarse and fine aggregates are stock piled at the site (Nos. 20 and 19 in Fig. 1) so as to have a continuous supply of the materials. The aggregates (Nos. 1 and 2) are stacked behind a bulkhead which has two openings to feed the coarse (No. 3) and fine aggregates (No. 4). The materials are taken up by a bucket elevator (No. 5) to the drier unit (No. 6). From the drier, the hot aggregates whose temperature can be recorded (No. 8) and also adjusted by

2

TABLE 9

**Gradation of designed mixes and limits**

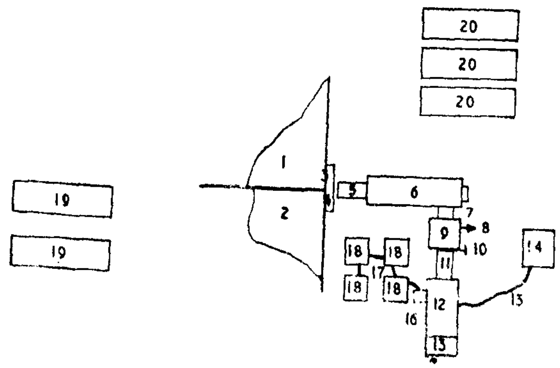
| Sieve size or No. | Lab. Mix 25 : 45 : 30 (Calculated from average gradations of ½ in., 3/8 in. and crusher dust) | | Field mix 2 : 3 : 2 by vol. or 27 : 44 : 29 by weight | | Specification limits Asphalt Institute mix No. IV b Manual Series No. 2 1956 |
|---|---|---|---|---|---|
| | Per cent by wt. retained | Per cent by wt. passing | Per cent by wt. retained | Per cent by wt. passing | Per cent by wt. passing |
| ¾ in. | 0.3 | 99.7 | 0.3 | 99.7 | 100 |
| ½ in. | 17.0 | 82.7 | 18.3 | 81.4 | 80–100 |
| ⅜ in. | — | — | — | — | 70–90 |
| No. 4 | 34.2 | 48.5 | 34.3 | 47.1 | 50–70 |
| ,, 8 | — | — | — | — | 35–50 |
| ,, 10 | 14.5 | 34.0 | 14.1 | 33.0 | — |
| ,, 30 | — | — | — | — | 18–29 |
| ,, 40 | 19.1 | 14.6 | 18.8 | 14.2 | — |
| ,, 80 | 8.6 | 6.0 | 8.4 | 5.8 | — |
| ,, 100 | — | — | — | — | 8–16 |
| ,, 200 | 3.7 | 2.3 | 3.6 | 2.2 | 4–10 |

adjusting the flame, pass to a hot bin (No. 9) from where again the quantity of hot aggregates fed can be controlled by a gate opening (No. 10). The aggregates ascend by a hot elevator (No. 11). Bitumen is supplied from four boilers (No. 18) through a pipe line (No. 17) and by means of a volumetric pump (No. 16) which is geared on to the pug mill (No. 12) in such a way that a predetermined quantity of bitumen can be sprayed on to the hot aggregates in the twin shaft pug mill, which is kept hot by steam jacketing, steam being obtained from a boiler (No. 14) and sent through a pipe (No. 15). The finished mixture can be loaded into tipper lorries through a jaw opening (No. 13). The machine as it is has four shortcomings, namely;



LAY OUT OF THE MIXING PLANT

NOT TO SCALE




LEGEND

| | | | |
|---|---|---|---|
| 1 | COARSE AGGREGATE STACK | 11 | HOT ELEVATOR |
| 2 | FINE AGGREGATE STACK | 12 | PUG MILL (STEAM JACKETED). |
| 3 | COARSE AGGREGATE CHUTE (ADJUSTABLE) | 13 | MIXTURE EXIT CHUTE |
| 4 | FINE AGGREGATE CHUTE (ADJUSTABLE) | 14 | STEAM ROAD ROLLER |
| 5 | BUCKET ELEVATOR FOR TOTAL AGGREGATE | 15 | STEAM LINE |
| 6 | DRIER WITH FLAME AND BLOWER | 16 | BITUMEN PUMP (STEAM JACKETED) |
| 7 | REVOLUTION COUNTER | 17 | BITUMEN PIPE LINE |
| 8 | TEMPERATURE RECORDER FOR AGGREGATE | 18 | BITUMEN BOILERS |
| 9 | HOT BIN | 19 | FINE AGGREGATE STOCK PILES |
| 10 | HOT BIN APRON GATE | 20 | COARSE AGGREGATE STOCK PILES |

Fig. 1

(a) there are only two openings through which the cold aggregates could be fed;

(b) there is no gradation control unit;

(c) there is no separate fines feeder unit; and

(d) some of the fines in the aggregates in the drier get blown off with the smoke on account of the strong draft.

3.4. Because of the above mentioned limitations of the mixing plant, a very rigorous design for the gradation control is not possible so as to be practicable. Therefore, the final design chosen should take into account the following considerations:

(a) Extra fines passing 200 mesh sieve cannot be added, not only from the point of view of cost but also from the point of view of plant limitations, and hence as much of fines as are available in the crusher run should be conserved by minimising the counter-weights in the exhaust blower of the drier.

(b) In as much as there are only two openings for feeding of the cold aggregates, the various sizes cannot be fed on separately and accurately so as to conform to the accepted specifications.

(c) In as much as there is no gradation control unit or weigh batching outfit, controls will have to be done on a volumetric instead of weight basis and regular periodical checks will have to be made.

3.5. Although the limitations of plant and cost may act as deterrents, yet from the laboratory point of view an attempt was made to see as to whether it would be possible to blend these aggregates so as to yield as dense a mixture as possible. Based on a series of trials in the laboratory, a proportion of 25 : 45 : 30 by weight for the ½ in., 3/8 in., and crusher run was arrived at and gradation given in Table 9. The specific gravity, as well as the loose and compacted bulk densities of the aggregates, were determined and are given in Table 7. Also, a series of mixtures using bitumen of varying quantities from 4.5 to 6.0 per cent were incorporated into this gradation and a series of Marshall and Hveem stability tests were conducted and test values obtained, Table 8 and Fig. 3. It can be seen from the gradation curve of this mixture, Fig. 2, that while the curve falls within the

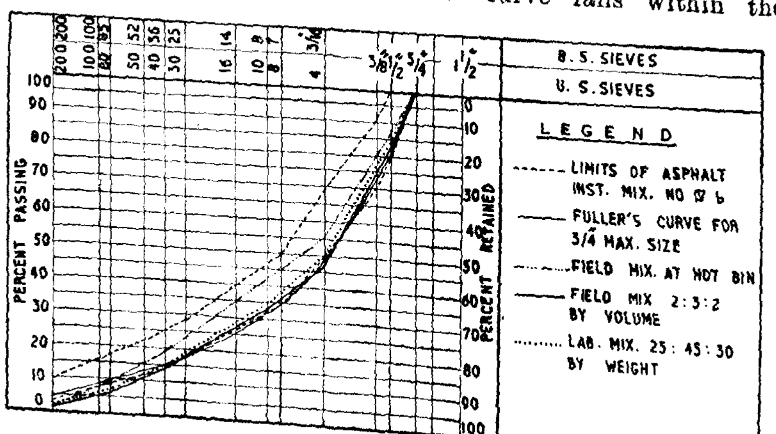



Fig. 2. Gradation curve



limits specified in the coarser sizes, it does not satisfy very strictly the limits in the finer ranges. It is also found that the percentage of fines is quite low and any attempt to improve the gradation would mean an addition of fine material which would not only mean extra cost but also necessitate modifications of the mixing plant itself, which may not be practicable. However,

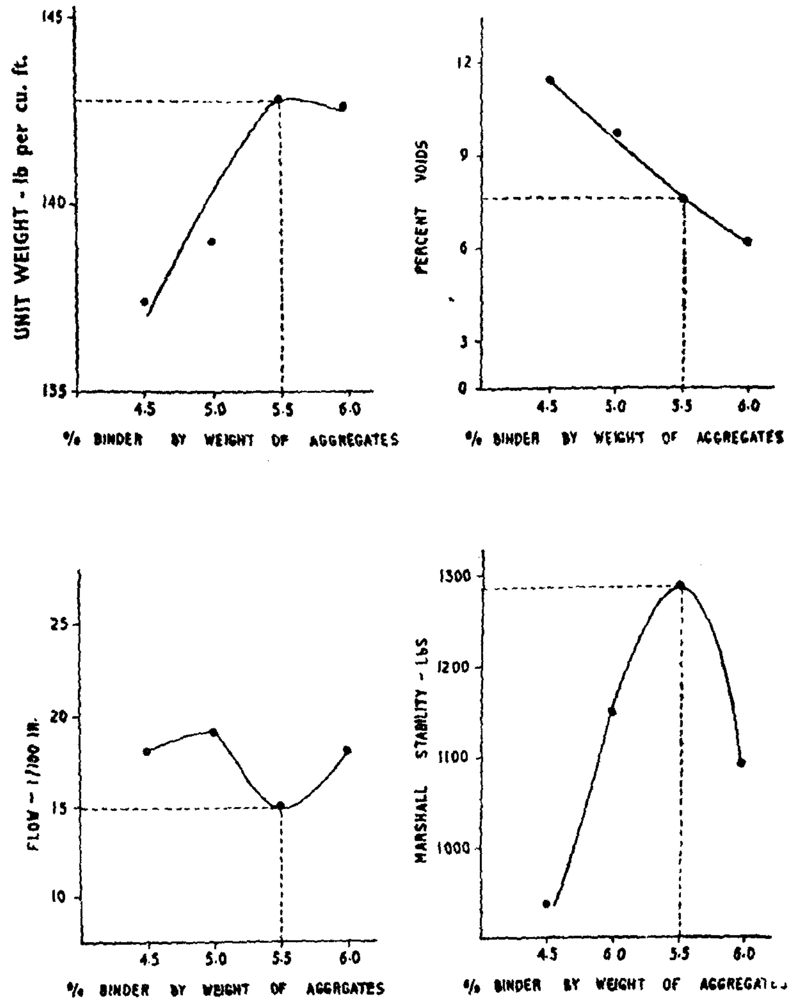



Fig. 3. PROPERTIES OF BITUMINOUS MIXTURE 25 : 45 : 30 BY WEIGHT

this is the grading which comes nearest to the specifications adopted[1]. From the point of view of stability, etc., it can be seen that with the proper quantity of binder it satisfies the stability as well as the void content requirements and as, such, should give a satisfactory working mixture, thereby corroborating what has been mentioned in para 3.1.

3.6. As already stated, since there was no weigh batching equipment attached to the mixing plant, mixing had to be done on a volumetric basis which implies that the various aggregates should be fed to the machine in terms of cubic feet based on bulk density data. It was found from the quarries that the normal volumetric yield from the crushers, Photo 15, is usually in the proportion of 2:3:2 for the ½: 3/8: crusher run. It was considered whether a mixture of the various ingredients in the same proportions in which they are obtained from the crushers would yield a satisfactory mixture or not. It appears reasonable that in as much as the three sizes are obtained by crushing 3 in. material, which is fed into the jaws of the crusher, it should be possible to get as dense a mixture or as near a solid volume as possible by mixing the three sizes in the same proportions in which they are obtained from the crusher. This 2:3:2 volume basis means a weight percentage of 27:44:29, the gradation of which is given in Table 9. It can be seen that this deviates only slightly from the originally chosen ratio but is easily adoptable for volumetric computations. However, such a mixture lacks in the finer ranges as a result of the blowing off of the fines at the crushers. This can also be seen in Fig. 2, wherein this gradation closely follows the Fuller curve for ¾ in. size in all sizes except in the fine range where deviations occur. Also, it was noted that the voids in the aggregate mixture, as a result of rodding alone, could be as low as 27 per cent.

## 4. CALIBRATION OF THE MIXING PLANT

4.1. It is obvious that the final gradation should be adoptable by the plant for which the formula for the job mix has to be ascertained. Since the mixing plant can accommodate only two sizes, it is suggested that the coarse aggregates, namely, ½ in. and 3/8 in., should be mixed in a volumetric ratio of 2:3 manually and then to feed the material through one of the two chutes and the crusher run through the second chute. A trial on the mixing plant using aggregates only was done and the chute openings at the bulk head were calibrated such that the volume of the coarse aggregates to the fine aggregates would be in the order of 5:2. Samples of the aggregate mixture were taken at three places, namely, at the cold bucket elevator, at the hot bin and at the pug-mill. The samples were taken at regular intervals



over a period of time and the average gradations of the aggregate mixture at the three places were obtained, Table 10. It appears

TABLE 10

**Gradation of the 2 : 3 : 2 mixture (dry) at various places**

| Sieve size or No. | (a) Cold bucket elevators | | (b) Hot bin | | (c) Pug mill | |
|---|---|---|---|---|---|---|
| | Percentage by wt. retained | Percentage by wt. passing | Percentage by wt. retained | Percentage by wt. passing | Percentage by wt. retained | Percentage by wt. passing |
| ¾ in. | 0.3 | 99.7 | 0.2 | 99.8 | 0.2 | 99.8 |
| ½ in. | 25.3 | 74.4 | 13.9 | 85.9 | 18.4 | 81.4 |
| No. 4 | 34.1 | 40.3 | 30.2 | 55.7 | 39.3 | 42.1 |
| ,, 10 | 11.2 | 29.1 | 12.8 | 42.9 | 12.7 | 29.4 |
| ,, 40 | 15.8 | 13.3 | 23.8 | 19.1 | 16.8 | 12.6 |
| ,, 80 | 6.1 | 7.2 | 11.1 | 8.0 | 7.5 | 5.1 |
| ,, 200 | 5.0 | 2.2 | 6.0 | 2.0 | 3.3 | 1.8 |

that there is a partial breaking down of the coarse aggregates, especially the ¾ in.— ½in. and ½ in.—No. 4 sizes from the bucket as it passes through the drier to the hot bin, thereby increasing the fraction on No. 4—No. 10, No. 10—No. 40 and No. 40—No. 80 sieves. But at the pug mill the percentage of coarse fractions is more, showing that there is a loss of fines especially of sizes No. 40—No. 80 and No. 80—No. 200 and passing No. 200, whileas the quantity of size No. 4—No. 10 remains more or less constant. The loss of this percentage of fines at the pug mill will be reduced obviously when binder is also incorporated since the pug mill is in a closed condition. It was also noticed that sampling at the buckets gave erratic results. Hence the composition of the hot bin mixture was taken as more or less representative of the final gradation of the aggregates in the mixture, though it is not nearest to the calculated proportion of the mixture.

4.2. It can be seen that this gradation more or less falls within the limits specified by the Asphalt Institute[1] though it may lack in the filler fraction. Also, this gradation was checked with the gradation modulus[3] as a tool and it was observed that the gradation modulus of the final gradation is 5.5 and as such it appears reasonable to expect a satisfactory mixture with the gradation chosen. Also, but for the filler fraction, it appears to conform to the specification laid in B.S. 594[3].

4.3. In order to determine the percentage of binder that would be necessary to give a satisfactory mixture for this gradation (the average of hot bin samples), a series of trial mixes were again made in the laboratory using 60/70 bitumen with various percentages of the binder from 4.5 to 6.5 (by weight on total dry aggregate) and the test results are given in Table 11. Also, Fig. 4 indicates that the optimum quantity of binder is 6 per cent on the dry aggregate basis, which means about 5.5 per cent on the total mix basis. This mixture has a void content of 7.4—9.4 depending on the method of compaction and

TABLE 11

**Test properties of the mixture 2 : 3 : 2 by volume ($\frac{1}{2}$ in. : 3/8 in. : cr. run) Bitumen 60/70 per cent by weight of total aggregates**

| Bitumen | 4.5 | 5.0 | 5.5 | 6.0 | 6.5 | Remarks |
|---|---|---|---|---|---|---|
| Marshall stability, lb (50 blows compaction) | 967 | 1130 | 1183 | 1209 | 1177 | Field mix. |
| Flow value - 1/100 in. | 22 | 17 | 15 | 18 | 20 | Gradation of the mixture |
| Percentage voids | 11.2 | 9.4 | 8.8 | 7.4 | 7.8 | As at hot bin, Table 10 |
| Unit wt. lb per cu. ft. (compacted) | 137.6 | 138.4 | 140.0 | 140.6 | 138.9 | |
| Hveem stability, percentage (compaction double plunger 3000 psi) | | 34.0 | 33.3 | 33.3 | | |
| Percentage voids | | 14.7 | 10.8 | 9.4 | | |
| Unit weight lb per cu. ft. (compacted) | | 135.9 | 136.2 | 137.5 | | |
| Swell value | | ... | ... | ... | | |

Photo 1. General view of the mixing plant

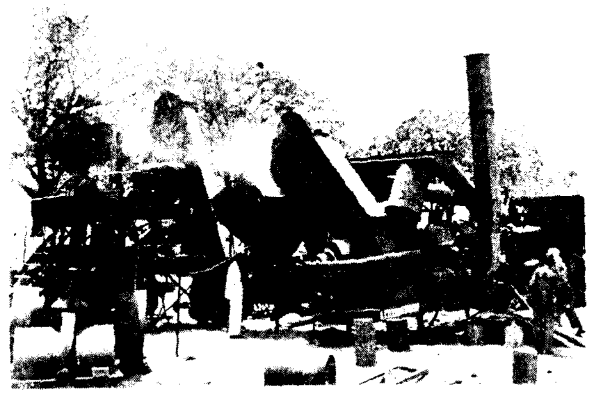


Photo 2. Hot bin view of the plant

Photo 3. Paver being loaded

Photo 4.
**Rear of paver**

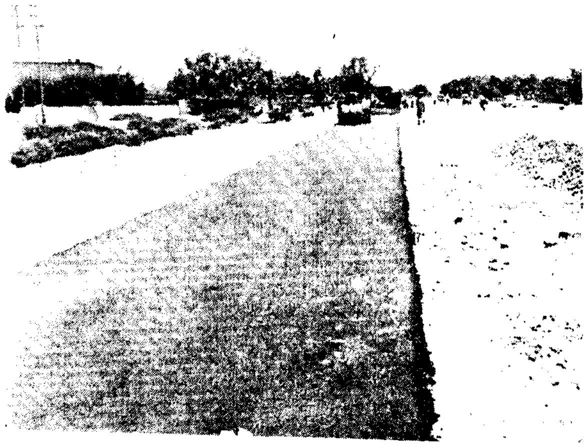


Photo 5.
**Rolling of the carpet**

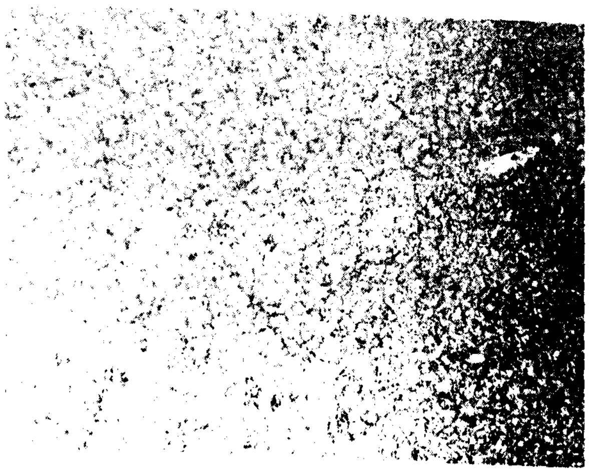


Photo 6.
**Texture of the finished surface**

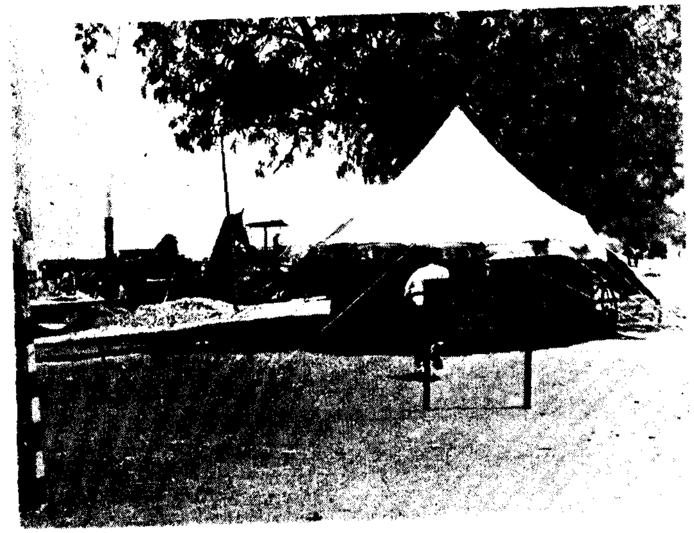


Photo 7.
**Field Laboratory**

Photo 8.
**Sampling at cold elevator**

Photo 9.
**Sampling at the hot bin**

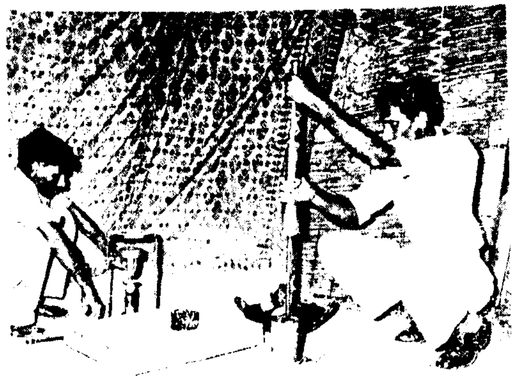


Photo 12.
**Moulding of test specimens**

Photo 10.
**Sampling of mix at pug-mill**

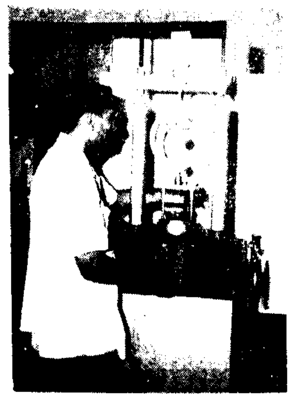


Photo 13.
**Marshall stability test**

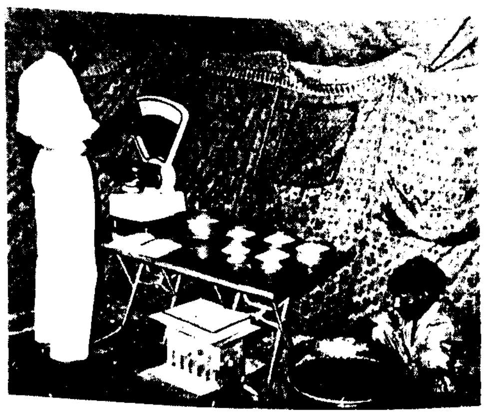


Photo 11.
**Sieve analysis of aggregates**

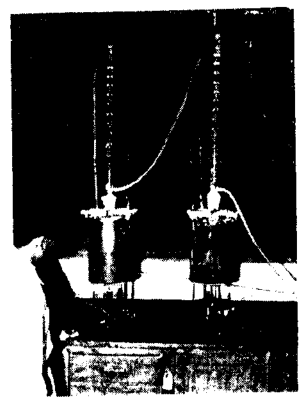


Photo 14.
**Extraction of binder**

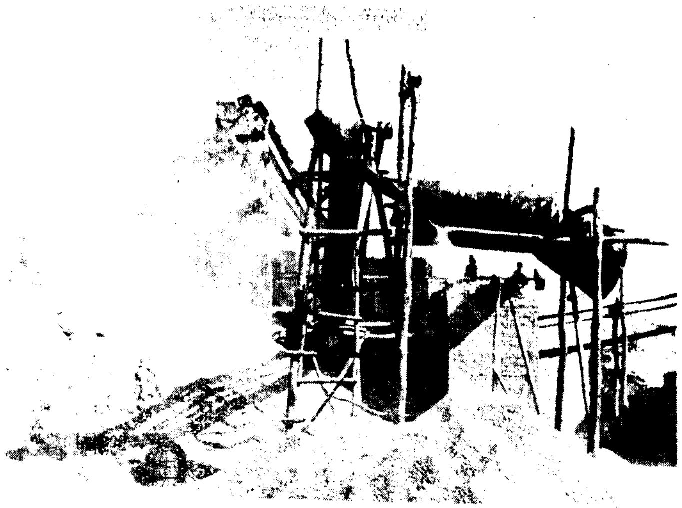


Photo 15.

Crusher with rotary sieves

Photo 16.

Storage bins of aggregates at the plant site



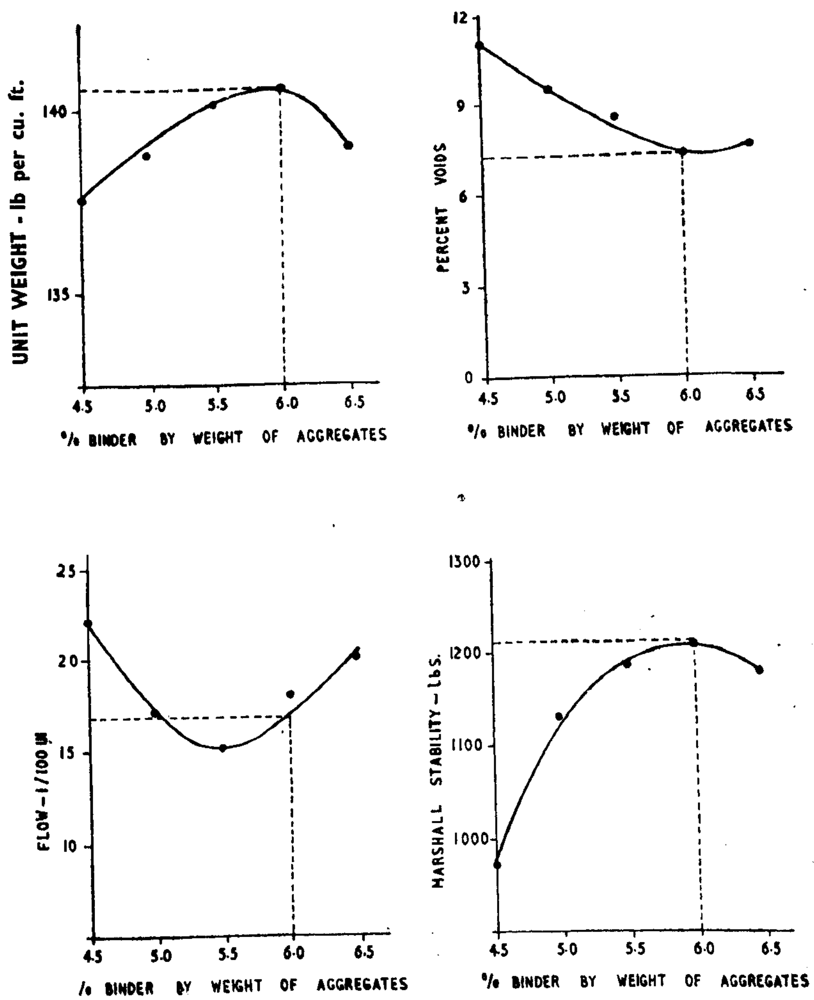



**Fig. 4. PROPERTIES OF BITUMINOUS MIXTURE 2:3:2 BY VOLUME.**

which is permissible in the case of graded asphalt mixtures as suggested by the Asphalt Institute.

4.4. As regards the nature of the binder, in order that the mix may meet the demands of high stress intensities on the road, it was thought that a harder grade of bitumen such as 60/70

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penetration grade would desirably be better than 80/100 as can be seen from the comparison of the stability data given in Table 12. The grade of 30/40 gives still better stability but

TABLE 12

**Test properties of the mixture 25 : 45 : 30 by weight (½ in. : 3/8 in. : Cr. run) with different grades of bitumen**

Per cent binder on total aggregate basis—5.5

| Bitumen | 80/100 | 60/70 | 30/40 |
|---|---|---|---|
| Marshall stability, lb (50 blows *compaction*) | 1286 | 1380 | 1840 |
| *Flow value 1/100 in.* | 15 | 20 | 18 |
| Percentage voids | 7.4 | 8.2 | 9.2 |
| Hveem stability, percentage (compaction by double plunger 3000 psi) | 36 | 37 | 40 |
| Percentage voids | 9.4 | 9.6 | 9.5 |

considering the climatic conditions and the compaction facilities, use of the grade 60/70 was preferred. Hence the mix combining ½ in., 3/8 in. and crusher run in the proportion of 2 : 3 : 2 with 5.5 per cent of 60/70 bitumen was considered as a proper design for the surface course mixture using Marshall method of design. This binder was supplied by bulk tank cars and stored in four boilers every day.

4.5. Having decided on the final gradation and the binder content, a trial mix was run again on the mixing plant by choosing the proper set of gear wheels for the bitumen pump so that a binder content of 5.5 per cent could be obtained for the mixture. Slight adjustments were done by controlling the gates at the hot bin before the aggregates move up the hot elevator to enter the pug-mill.

## 5. LAYING OF THE CARPET

5.1. The bituminous mixture coming out of the pug-mill normally at a temperature of 300°F. is loaded directly into tipper trucks and taken to the site where the carpet is to be laid. The



mixture remains hot for more than an hour even, during which time the trucks take the material to the paver. Before the mix is actually laid, the bed is prepared with a tack coat of bitumen so as to ensure proper bondage between the bed and the new carpet. In as much as the water-bound macadam base was new and in order to have adequate adhesion between the base and the carpet, the carpet being as thin as 1 in., it was decided to give a tack coat of hot bitumen 60/70 at the rate of 20 lb per 100 sq. ft. and to lay the tack just immediately before the paver went over the base.

5.2. The paver consists of two main units, the tractor unit and the screed unit, Photo 3. The tractor unit provides the motive power through the crawlers and this unit includes the receiving hopper, feeders, spreading screws and power plant. The screed unit consists of the tamper, thickness controller and screed. The material received in the hopper is taken by a movable apron past the gates and goes into the screws. The spreading screws uniformly mix and spread the material to the full width of 11 feet of the surface. Behind the screws are the tampers whose height can be adjusted and which strikes the material and compacts the mixture. The screed which is immediately behind completes the smoothing and ironing of the surface with the result that as the tipper lorry feeds the hopper and as the paver moves at a speed of 15 to 20 ft per minute, it leaves behind a more or less compacted bituminous surface appearing as if a carpet is being unrolled, Photo 4.

5.3. Since the carpet is still hot and has not received complete compaction, smooth wheel 10 ton road rollers are put over it to give the final compaction of the mixture, Photo 5.

5.4. Although in the laboratory one may get as low a voidage by changing over the method of compaction, it is necessary to know the amount of rolling that would be required so that as dense a carpet as possible can be obtained in the field for a given mixture. With this object in view, since the department supplies only a three wheel roller and to get a correlation between laboratory and field densities five lengths of 100 ft each were chosen immediately after the paver had laid the carpet and in each of the 100 ft, the number of passes of a 10 ton roller was changed from no rolling to 4 passes of the roller. The field densities were determined and it was found, Table 13, that the maximum density was obtained when the number of passes was between 3 to 4, and this was stipulated as the amount of rolling required for the carpet.

TABLE 13

**Field density versus number of passes of roller**

| | Density (*average*) |
|---|---|
| No rolling | 1.60 |
| One pass | 1.79 |
| Two passes | 1.86 |
| Three passes | 1.98 |
| Four passes | 1.97 |

TABLE 14

**Unevenness measurement with bump integrator**

Width : 22 feet Speed of travel : 20 m.p.h.

| No. | Side | Over the water-bound macadam base | | | Over the carpet | | |
|---|---|---|---|---|---|---|---|
| | | Wheel revolution | Bumps recorded | Index in./mile | Wheel revolution | Bumps recorded | Index in./mile |
| 1 | Left | 451 | 143 | 234 | 693 | 104 | 112 |
| 2 | Right | 456 | 127 | 206 | 688 | 95 | 102 |
| 3 | Centre | 426 | 127 | 220 | — | — | — |

*N.B.*—Normally the limits specified are as under :

Below 130 — Good or better

130 – 180 — Fairly good.

5.5. In order to evaluate the smoothness of the carpet after it has been laid and to note the degree of betterment as a result of this carpet, a bump integrator was run before and after the carpet was laid over the base. Table 14 indicates that as a result of proper design and mechanical laying, a smooth bump-free surface would be available, Photo 6—a result which cannot easily be obtained with ordinary manual laying.



TABLE 15

**Central Road Research Institute**

**Design of asphaltic concrete mixture for Delhi-Mathura Road (by-pass)**

Date...15-7-'59 Weather...Clear Location : By-pass towards Rajghat from Mathura Road.
Day...Wednesday Atm. temp...12 Noon 93°F. Northern carriageway,
Northern half starting from
3 P.M. 100°F. Barapula Bridge end to Rajghat
532 ft.
Checked by.....



| S.No. / Sampled at : / Counter No. | 66 2 [illegible] 3 mix : chute 1.25 P.M. | | | 62 Crusher run : chute 1.25 P.M. | | | H 41 Hot-bin 2.00 P.M. 112680 | | | P-41 pug-mill 2.05 P.M. 112700 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Percentage ret. | Cum. percentage ret. | Percentage passing | Percentage ret. | Cum. percentage ret. | Percentage passing | Percentage ret. | Cum. percentage ret. | Percentage passing | Percentage ret. | Cum. percentage ret. | Percentage passing. |
| | | | | | | | | | | — | — | 100 |
| 1 in. – ¾ in. | 1.6 | 1.6 | 98.4 | — | — | — | 0.4 | 0.4 | 99.6 | 16.2 | 16.2 | 83.8 |
| ¾ in. – ½ in. | 26.0 | 27.6 | 72.4 | — | — | — | 16.0 | 16.4 | 83.6 | 11.5 | 27.7 | 72.3 |
| ½ in. – ⅜ in. | 16.6 | 44.2 | 55.8 | — | — | 100 | 11.0 | 27.4 | 72.6 | 18.2 | 45.9 | 54.1 |
| ⅜ in. – No. 4 | 26.6 | 70.8 | 29.2 | 1.5 | 1.5 | 98.5 | 19.8 | 47.2 | 52.8 | 14.5 | 60.4 | 39.6 |
| No. 4 – No. 10 | 17.7 | 88.5 | 11.5 | 2.2 | 3.7 | 96.3 | 14.3 | 61.5 | 38.5 | 22.8 | 83.2 | 16.8 |
| No. 10 – No. 40 | 8.1 | 96.9 | 3.1 | 53.6 | 57.3 | 42.7 | 23.3 | 84.8 | 15.2 | 9.5 | 92.7 | 7.3 |
| No. 40 – No. 80 | 3.1 | 100 | — | 21.7 | 79.0 | 21.0 | 7.3 | 92.1 | 7.9 | 4.5 | 97.2 | 2.8 |
| No. 80 – No. 200 | — | — | — | 14.5 | 93.5 | 6.5 | 5.5 | 97.6 | 2.4 | 2.8 | 100 | — |
| Passing No. 200 | — | — | — | 6.5 | 100 | — | 2.4 | 100 | — | | | |

**Bitumen percentage :**

**Total mix.........5.77**

**Total agg.........6.12**

**Mix laid at.................................200 ft. from start.**

**Remarks : Marshall stability test specimen was made from pug-mill mixture and same mixture sent to CRRI for extraction of binder and sieve analysis of the aggregates.**

TABLE 16

**Central Road Research Institute**

**Field mix data**

Project—Mathura Road By-pass—(towards Rajghat)

| Date & Day | Time of sampling | Sampled from | Laid at | Mix temp. | S. No. specimen | Tested on | Bulk sp. gr. | Theor. sp. gr. | Percentage voids | Marshall stability lb Obs: | Marshall stability lb Corrected | Flow value | Remarks: Test results of bitumen used Penetration | Remarks: Test results of bitumen used Softening point |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 15-7-'59 Wednesday | 2.05 P.M. | Pug-mill | By-pass | 300°F. | P-41 | 16-7-59 | 2.242 | 2.432 | 7.8 | 910 | 1083 | 18 | 63 (bulk supply) | 52.6 |



## 6. FIELD CONTROL

With a view to ensure proper gradation, mixing and laying and for quality control of the paving mixture, a field laboratory was set up very near the plant, Photo 7, and the following checks were made :

### 6.1. Material Control

1. The sieve analysis of the 2 : 3 mixture for checking of the proportioning of the coarse aggregates, Photo 16.
2. Sieve analysis of the crusher run for variations, if any.
3. Sieve analysis of the final aggregate mixture of 2 : 3 : 2 at the cold elevator and again at the hot bin, Photos 8 & 9, so as to record the composition of the dry mixture from time to time and to make suitable adjustments in the proportions whenever necessary. The adjustments were made by raising or lowering slightly the gates at the cold elevator.
4. Samples of bitumen taken from bulk tank cars or drums as the case may be for penetration and softening point tests.
5. Samples of the final bituminous mixture obtained, Photo 10, from the pug-mill periodically to conduct tests such as extraction of the binder and the gradation analysis of the aggregates. Also, test specimens were moulded with this mixture and the stability, void content, etc. of the specimens were determined, Photos 11, 12, 13 & 14.

These above checks were done initially over every hour or more often when required and gradually reduced. The results of the control tests were recorded as shown in the two Tables 15 and 16 and any major variation found in the gradation of the materials, stability value and binder content of the mixture were recorded immediately and reported to the field staff for necessary adjustments.

### 6.2. Plant Control

1. Recording of the temperature of the material at the hot bin as well as the bitumen in the boilers to avoid over or under heating of the material and to take

necessary steps to adjust the flame in the burners wherever necessary for a temperature of 350—375°F. for the bitumen and 250—275°F. for the aggregates.

2. Check on the regular and continuous flow of the materials through the chutes so that the flow may be according to the requirements.
3. Check on the uniformity of coating in the finished mixture.
4. Recording of the counter number with the hot bin at the start and at the end of the day's work to calculate the quantity of materials used in a day.
5. The counter number as each tipper lorry moved out of the plant so as to locate the actual site where the mixture was laid from time to time.

### 6.3. Laying Control

1. Cleaning of water-bound macadam and application of the hot bitumen for tack coat over the dry base.
2. Edge of the previous day's carpet to be cut and binder applied for joints.
3. Adjustment of paver for thickness of carpet and the thickness of carpet laid checked periodically.
4. Mixture to be laid before it is cooled—the temperature of the mixture being not less than 250°F.
5. Removal of loose stones and oversizes and dressing of the laid mixture by filling finer material over places rich in coarse aggregate as a result of segregation or improper gradation or defect in laying.
6. Roller to pass longitudinally three to four times at a speed not exceeding 2 m.p.h. when the mixture is partially cooled down.
7. To check the longitudinal joints and camber.
8. Measurement of the carpet area done every day to calculate the quantities of materials used.

In addition to the above, the daily temperature and climatic conditions were also recorded. Again, as a check over the degree of compaction obtained in the field, field densities in certain chosen spots were also done, Table 18, to see whether the mix has been compacted to the desired extent or not.



TABLE 17

**Test properties of the mixture (2 : 3 : 2) versus variation in binder content and gradation**

(Per cent by weight retained on each sieve used)

| Sieve size or No. | Required gradation | | Gradation of mixture after extraction of binder | | |
|---|---|---|---|---|---|
| | Design | Hot bin | Normal | Normal | Coarse |
| 3/4 in. | 0.3 | 0.2 | – | – | 0.9 |
| 1/2 in. | 18.3 | 13.9 | 14.4 | 16.9 | 23.0 |
| No. 4 | 34.3 | 30.2 | 32.3 | 29.5 | 16.7 |
| No. 10 | 14.1 | 12.8 | 14.5 | 14.1 | 12.4 |
| No. 40 | 18.8 | 23.8 | 19.4 | 19.9 | 27.3 |
| No. 80 | 8.6 | 11.1 | 11.9 | 12.1 | 11.7 |
| No. 200 | 3.6 | 6.0 | 5.3 | 5.3 | 6.2 |
| Passing No. 200 | 2.2 | 2.0 | 2.2 | 2.2 | 1.8 |
| *Binder Content* | | | *Normal* | *Low* | *Normal* |
| Total mix basis | | | 5.7 | 4.1 | 5.7 |
| Total aggregate basis | | | 6.0 | 4.3 | 6.0 |
| Marshall stability lb | | | 1202 | 556 | 623 |
| Flow value 1/100 in. | | | 16 | 14 | 20 |
| :Percentage voids | | | 8.3 | 12.5 | 11.3 |

TABLE 18

**Field density of laid carpet**

| Location No. | Density (bulk) | Theoretical * density | Percentage voids | Thickness of carpet, in. |
|---|---|---|---|---|
| 1. | 2.170 | 2.490 | 12.8 | 1½ |
| 2. | 2.130 | 2.423 | 12.0 | 1¼ |
| 3. | 2.000 | 2.438 | 18.0 | 1 |
| 4. | 2.090 | 2.429 | 13.9 | 1¼ |

* Theoretical density is calculated after extraction of binder and from the percentage composition of the mixture after extraction.

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## 7. OPERATIONAL PROBLEMS

Because of the degree of mechanisation involved and the fast pace of working, it is but natural that there will be some operational problems which are to be faced and overcome during the laying of the carpet. The following are some of the problems that were confronted during the laying of the stretch, discussed in this Paper.

### 7.1. Bitumen

It is highly important that the correct quantity of bitumen should be used for the mixture so as to ensure sufficient strength of the pavement. An example is given in Table 17 where the binder content fell far below the limit and the strength obtained in the mix as a result of this lower quantity of binder employed. Normally, when the binder content deviates only slightly, it can be corrected, based on the extraction test data, by varying the opening of the gate at the hot bin before the aggregates are sent up the hot elevator. If, on the other hand, the variation is very high, then the check will have to be made on the nozzle and the bitumen pump and, if need be, the nozzle will have to be replaced or even changes in the gearing ratio of the pump may have to be resorted to. Just as the binder content is to be checked periodically, the temperature of the binder also will have to be checked so that the binder does not get overheated with a consequent loss of adhesion. A lower temperature of the binder would obstruct the free flow and thereby affect the coating and also quantity of bitumen supplied.

### 7.2. Aggregates

In a bituminous mixing plant without weigh-batching provisions, variation in the gradation is bound to occur unless strict control is enforced. Examples given in Table 17 show the effect on strength due to gradation changes and Table 19 indicates the degree of variations possible in the aggregate mixture during a day and stresses the absolute necessity for a gradation control unit in a work such as this.

It was noticed that the variation in the gradation occurred mainly as a result of manual mixing of the ½ in. and 3/8 in. aggregates. So, to minimise the variation it was suggested that the two aggregates should be spread evenly in the storage bin such that there are three layers of 3/8 in. and two layers of ½ in. aggregate interspersed between one another, with the result that as the labour dig the spade to the full depth into the aggregate bin for loading of the baskets, mixing takes place automatically.



TABLE 19

**Typical variation during a day in the gradation of the mixture 2 : 3 : 2 at the plant**

(Percentage by weight retained on each sieve)

| Sieve size or No. | Design gradation | Hot bin gradation | Hot bin mixture gradation | | | |
|---|---|---|---|---|---|---|
| | | | Satisfactory | Coarse | Fine | Coarse with over size |
| 3/4 in. | 0.3 | 0.2 | 0.4 – 1.8 | 1.5 – 4.2 | – | 3.5 – 10.3 |
| 1/2 in. | 18.3 | 13.9 | 13.3 – 16.0 | 22.9 – 31.7 | 6.5 – 8.0 | 18.3 – 25.4 |
| No. 4 | 34.3 | 30.2 | 30.6 – 30.8 | 26.6 – 32.7 | 19.7 – 24.4 | 24.4 – 28.4 |
| No. 10 | 14.1 | 12.8 | 14.0 – 14.3 | 8.3 – 10.0 | 13.6 – 16.1 | 7.6 – 10.2 |
| No. 40 | 18.8 | 23.8 | 23.3 – 23.9 | 15.8 – 19.2 | 27.8 – 28.1 | 18.3 – 22.8 |
| No. 80 | 8.6 | 11.1 | 7.3 – 8.2 | 6.2 – 8.3 | 12.0 – 14.3 | 6.5 – 8.6 |
| No. 200 | 3.6 | 6.0 | 5.5 – 5.6 | 4.4 – 5.1 | 10.0 – 11.3 | 5.3 – 7.0 |
| Passing No. 200 | 2.2 | 2.0 | 2.4 – 2.6 | 1.5 – 1.6 | 3.6 – 4.2 | 1.7 – 2.1 |

However, periodical checks and vigilant control are necessary to lessen the degree of variation of the gradation.

Because of moisture in the crusher run, quite often there is no free flow of the material through the chute near the bucket elevators, which, if not checked at the proper time, would give a mix devoid of the fine fractions. The aggregates again have to be heated to the required temperature and also it should be seen that the flame consumes all the fuel that is supplied to it. This can be checked by seeing the colouration of the smoke which indicates complete or incomplete combustion.

### 7.3. Segregation

The amount of segregation of the coarse from the fines would depend upon the number of times the mixture is handled and moved about. Segregation within the mixture can normally occur as the mix is dumped into the receiving truck. The coarse aggregates usually travel down the cone pile and accummulate at the corners of the truck. When this segregated mixture is dumped into the spreader, sometimes remixing is done by the screws while spreading. A solution would be to see that the segregated material is thrown back into the centre of the moving apron in the hopper of the spreader.

### 7.4. Tack Coat

A tack coat using hot bitumen was used during the construction. It was found that the trucks which move in front of the paver for loading of the paver pick up the base as a result of the freshly laid tack coat. The way by which this was overcome was to see that sufficient length of the tack coat was laid in front of the paver only after the trucks have unloaded themselves into the paver. Another solution may be to prime the entire surface a day in advance or to have a fluid type of rapid curing cut-back from an overhead tank and which can be sprayed on just before the laying of the carpet. The tack coat might have been avoided if the thickness of the carpet was atleast 2-3 in. or if the carpet was laid over an existing blacktop surface, or, alternatively, if the base had been given a bituminous primer a few days in advance.

### 7.5. Laying

It is obvious that any flaw in the paving mechanism would affect the smoothness of the carpet. The three gates, which control the rate of flow of the mixture from the hopper into the screws, should be checked to see that the material flows uniformly to the screws thereby lessening the segregation of the mixture. It is also obvious that if the paver is driven over loose



stones, unevenness of the carpet would result as a result of varied thicknesses.

### 7.6. Rolling

Adequate rolling should be resorted to and this should not be postponed till the mixture cools down completely nor should the roller pass immediately behind the paver when the mixture is still hot. The number of passes that has been stipulated, based on the initial field tests, should be resorted to so that adequate compaction is ensured. And at places inaccessible to the roller, hand tamping is a must.

### 7.7. Thickness of the Carpet

When the thickness of the carpet is limited to 1 to $1\frac{1}{2}$ in., it is but obvious that unless the base has been prepared well and formed to camber with due regard to levels, etc., the thickness will be more than $1\frac{1}{2}$ in. and may even be less than 1 in. depending on the condition of the base. But since the carpet is thin, it is emphasised that in no place over the road should the thickness be less than 1 in. Where the base is formed by manual labour without resorting to accurate controls and levels, by using boning rods, etc., and when the tolerance in the base is more than $\frac{3}{8}$ in., the quantities required for the construction may have to be worked out for a higher thickness so as to cater to the variations in the quantities as a result of inequalities of the base.

The quantities required for a given thickness should be worked out by taking into consideration the bulk densities of the various sizes and of the mix as given in Appendix I.

### 7.8. Co-ordination

Although the overall construction demands only a minimum number of skilled workmen to operate the various machinery, it is absolutely essential that complete coordination should exist between the supply of materials, the operators of the mixing plant, the operators of the trucks and the operators of the paver and the laying of the tack coat. A break in the chain or even a delay or improper timing would lessen the speed of construction and thereby increase the total cost. The cost structure for the work is given in Appendix II. During the construction, owing to the breakdown of machinery, insufficient number of tippers for places with long leads, delay in supply of binder, etc., the pace of work varied from 2 hours per day to 6 hours per day, as a result of which the area covered was from 7,000 sq. ft. to 17,000 sq. ft. per day. Taking into account the distance from the mixing plant to the paver, the required number of tippers or trucks should be provided so that the mixture as it continuously flows from the plant into the truck flows into the paver with as little a delay as possible and the work can only then move on even keel.

### 7.9. Control

In a work such as this it is imperative that the control laboratory should be established at the site not only to control the quantity and quality of the materials that are fed into the mixer but also to control the mixture and the subsequent operations to see that the finally compacted carpet would possess the same strength and density as was originally designed. More than the actual laboratory testing itself, it is the execution and implementation based on the scientific laboratory data that is of paramount importance, since the main purpose of the control laboratory is only to indicate as to when and where the things go wrong.

## ACKNOWLEDGMENT

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The Authors wish to record their thanks in particular to the Consulting Engineer (Roads Development) who made it possible for the project to be taken up on a scientific level.

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## QUANTITIES OF MATERIALS REQUIRED

### Appendix I

The quantities of various materials required per 100 sq. ft. for 1 in. compacted thickness would be calculated as shown below :

The unit weight of the compacted mixture = 140.5 lb per cu. ft.

So, for 100 sq. ft. for thickness of 1 in.

quantity of mixture required $= 100/12 = 25/3$ cu. ft.

On weight basis quantity $= 25/3 \times 140.5 = 1170$ lb

Percentage of binder in the mix = 5.5

So, quantity of binder for 100 sq. ft. $= \dfrac{1170 \times 5.5}{100} = 64$ lb

So, weight of aggregates = 1106 lb

On a weight proportion of 27:44:29 for the ½ in. : 3/8 in. : Cr. run

Quantity of ½ in. aggregate $= \dfrac{1106 \times 27}{100} = 298$ lb

Quantity of 3/8 in. aggregate $= \dfrac{1106 \times 44}{100} = 487$ lb

Quantity of crusher run $= \dfrac{1106 \times 29}{100} = 321$ lb

Based on loose bulk density of 82.7 lb per cu. ft. for ½ in. aggregate quantity required per 100 sq. ft. = 3.6 cu. ft.

Based on compacted bulk density of 95.5 lb. per cu. ft. for ½ in. aggregate quantity required = 3.1 cu. ft.

Similarly based on loose and compacted bulk densities of 89.4 lb per cu. ft. and 102.5 lb per cu. ft. for the 3/8 in. aggregates, quantities required are 5.4 and 4.7 respectively.

Similarly based on loose and compacted bulk densities of 87.5 and 103.1 for the crusher dust, quantities required are 3.6 and 3.1 cu. ft.

Hence the quantities of the materials required for 100 sq. ft. for a compacted 1 in. thick carpet :

(a) Bitumen (i) 65 lb for mix
(ii) 20 lb for tack = 85 lb

(b) Aggregates
(i) ½ in. metal—3.1-3.6 say 3.25 cu. ft.
(ii) 3/8 in. metal—4.7-5.4 say 5.5 cu. ft.
(iii) Crusher run—3.1-3.6 say 3.25 cu. ft.

## Appendix II

### DESIGNED CARPET

1 in. designed mix carpet using 12 cu. ft. material or 3.25 cu. ft. of $\frac{1}{2}$ in. aggregates, 5.25 cu. ft. of 3/8 in. and 3.5 cu. ft. of crusher run with 65 lb of bitumen 60/70 and a tack coat of 20 lb of bitumen 60/70 per 100 sq. ft.

(Note: 3.25 cu. ft. of $\frac{1}{2}$ in. aggregate and 5.25 cu. ft. of 3/8 in. after intimate mixing will lose 6 per cent by volume and give 8 cu. ft. approx).

**Details for 10000 sq. ft.**

| | Quantity per 10000 sq. ft. | Rate Rs | Cost per 10000 sq. ft. Rs | Remarks |
|---|---|---|---|---|
| **I. Bitumen** | | | | |
| (a) For carpet at 65 lb per 100 sq. ft. $= \frac{65 \times 10000}{100 \times 2240}$ | 2.9 tons | | | |
| (b) For tack coat at 20 lb per 100 sq. ft. $= \frac{20 \times 10000}{100 \times 2240}$ | .89 ton | | | |
| | 3.79 tons | 286 per ton | 1083.94 | |
| **II. Aggregate** | | | | |
| (a) Supplying and stacking D.Q. stone grit $\frac{1}{2}$ in. and 3/8 in. aggregates mixed in the proportion 2 : 3 inclusive of loading at the mixer $\frac{8 \times 10000}{100}$ | 800 cu. ft. | 48 per 100 cu. ft. | 384.00 | |
| (b) Supplying and stacking of crusher run D.Q. stone grit inclusive of loading at the mix $\frac{3.5 \times 10000}{100}$ | 350 cu. ft. | 28.27 per 100 cu. ft. | 98.95 | |
| **III. Labour** | | | | |
| (a) For cleaning as per schedule | 3/4 @ 2.62 – 1.97<br>13 @ 2.00 – 26.00<br>13 @ 1.75 – 22.75 | | 50.72 | |



| | Rs. |
|---|---|
| (b) For heating of bitumen beldar: $4 \times 3.79 = 15.16$ say 15 @ Rs 2 each | 30.00 |
| (c) For tack coat as per schedule beldars $6\frac{1}{2}$ @ Rs 2.00 | 13.00 |
| (d) Chowkidar $2\frac{1}{2}$ @ Rs 2.00 = 5.00<br>Bhisti 1 @ Rs 2.62 = 2.62 | 7.62 |
| **IV. Steam coal** | |
| (a) For roller = 6 cwt. 6 cwt.<br>(b) For boiler $4 \times 3.79$ 15.16 cwt. } Rs. 2 per md. | 64.91 |
| (c) Carriage of above 21.16 11 nP per. cwt. | 2.33 |
| **V.** Hire charges for roller @ Rs 18.00 per day | 18.00 |
| Brushes | 2.50 |
| Sundries for brooms, gunny bags etc | 5.00 |
| Plant charges for the mixing and laying for 10,000 sq. ft 4 per 100 sq. ft. | 400.00 |
| Total | 2160.97 |
| Miscellaneous contingencies and profits about 10 per cent | 216.09 |
| | 2377.06 |
| Say | 2400.00 |
| Rs 24 per 100 sq. ft. | |

Since it is stipulated that the thickness of the carpet should be a minimum of 1 in. and in order to absorb the unevenness of the W.B. base, the quantity of materials could be increased by 25 per cent, in which case the material used would be 10 cu. ft. of $\frac{1}{2}$ in. and 3/8 in. mixed aggregates together with 4.5 cu. ft. of crusher run for 100 sq. ft. The total cost for 10,000 sq. ft. would then be

$$= 1.1 \left\{ \begin{array}{l} 2160.97 + \frac{1}{4}(2.9 \times 286 + 384 + 98.95 + 400) \\ \qquad + \frac{2.9}{4} \times 4 \times 2 + 2.9 \times \frac{11}{100} + 4.16 \times 2 + 1 \end{array} \right\}$$

$= 2753.85$ or say Rs 28 per 100 sq. ft.

## Appendix III

### OPEN GRADED PREMIX CARPET WITH SEAL COAT

1 in. premix carpet using 10 cu. ft. of stone grit ½ in. and 35 lb of hot bitumen for premix and 20 lb of hot bitumen for tack coat for 100 sq. ft. of surface together with a seal coat using 20 lb of hot bitumen and 3 cu. ft of ¼ in. grit for 100 sq. ft.

### Carpet for 10,000 sq. ft.

| | Quantities | Rate | Cost | Remarks |
|---|---|---|---|---|
| **I. Bitumen** | | | | |
| (a) For carpet at 35 lb 100 sq. ft. $\frac{35 \times 10{,}000}{100 \times 2240} =$ | 1.6 tons | | | |
| (b) For tack coat $\frac{20 \times 10{,}000}{100 \times 2240}$ | 0.9 | 380 per ton | 1292.00 | |
| (c) For seal coat | 0.9 tons | | | |
| | 3.4 | | | |
| **II. Aggregates** | | | | |
| (a) Supplying of ½ in. stone grit | 1000 cu. ft. | 48/100 | 480.00 | |
| (b) Supplying of ¼ in. grit | 300 cu. ft. | 50/100 | 150.00 | |
| **III. Labour** | | | | |
| (a) For cleaning as per schedule | | | 50.72 | |
| (b) Heating bitumen 4×3.4 @ Rs 2.00 | | | 27.20 | |
| (c) For tack as per schedule | | | 13.00 | |
| (d) Screening—mistry 1¾ @ Rs 5.25 | | | 9.19 | |
| Beldars $\frac{5.25 \times 1300}{100}$ 68 @ Rs 2.00 | | | 136.00 | |
| (e) Miscellanous as per schedule | | | 7.62 | |
| | | | 2165.73 | |
| **IV. Fuel** | | | | |
| (a) For roller at 6 cwt. (b) For boiler 4×3.4=13.6 | 19.6 cwt | Rs. 2 per md. | 56.20 | |
| (c) Cartage at 11 nP per cwt. | | | 2.16 | |
| **V. Plant hire charges** | | | | |
| Road roller 18.00 per day; mixer 6.00 per day; boiler 6.00 per day; brushes 1.25 per day | | | 37.50 | |



| | | |
|---|---|---|
| Broom, gunny bags, sundries. 4.00 per day | | |
| | Total | 2261.59 |
| Add 10 per cent for profits etc. | | 226.15 |
| | Net total | 2487.74 |
| | Say Rs 25 per 100 sq. ft. | |
| Cost of seal coat only | Rs 6 per 100 sq. ft. | |

If bulk bitumen and the mixing plant together with the paver is used, the cost per 100 sq. ft. would work out as:

$$= 2487.74 - (319 + 136 + 9.19 + 37.5) + 400 + 25.5 = 2411.5$$

say Rs 24 per 100 sq. ft.

## Appendix IV

### STONE-SAND ('CRETE') CARPET

1 in. bituminous concrete carpet with 8 cu. ft. of stone chips (¾-½ in. 50 per cent, 3/8 in. 25 per cent and ¼ in. 25 per cent) with coarse sand of 4 cu. ft. for 100 sq. ft. with hot bitumen at 4 lb per cu. ft. of stone chips and 8 lb per cu. ft. of sand together with a tack coat of 20 lb of hot binder per 100 sq. ft of surface.

### Details for 10,000 sq. ft.

| | | Quantity | Rate | Cost |
|---|---|---|---|---|
| **I. Bitumen** | | | | |
| Mix and tack $\frac{(8 \times 4 + 8 \times 4 + 20) \times 100}{2240}$ | | 3.74 ton | 380 ton | 1421.20 |
| **II. Aggregates** | | | | |
| i. ¾-½ in. stone grit | 4×100 | 400 cu. ft. | 48 | 192.00 |
| ii. 3/8, ¼ in. stone grit | 4×100 | 400 cu. ft. | 50 | 200.00 |
| iii. Sand—crusher run | 4×100 | 400 cu. ft. | 28.27 | 113.08 |
| **III. Labour** | | | | |
| (a) For cleaning as per schedule | | | | 50.72 |
| (b) For bitumen 4×3.74 @ Rs 2.00 | | | | 29.92 |
| (c) For tack coat as per schedule | | | | 13.00 |
| (d) For screening mistry 1¾ at 5.25 | | | | 9.19 |
| Beldar $\frac{5.25}{100} \times 1200$ i.e. 63 @ Rs 2 | | | | 126.00 |
| (e) Miscellaneous as per schedule | | | | 7.62 |
| **IV. Fuel, etc.** | | | | |
| (a) For roller 6 cwt. | | | | |
| (b) Boiler—14.96 | | 20.96 cwt. | Rs 2 per md. | 57.20 |
| (c) Cartage | | 20.96 at 11 nP per cwt. | | 2.30 |
| **V. Hire charges for roller, etc.** | | | | |
| As per schedule | | | | 37.50 |
| Total | | | | 2259.73 |
| Add 10 per cent for profit | | | | 225.97 |
| Net total | | | | 2485.97 |
| Say Rs | | | | 25 per 100 sq. ft. |

With bulk bitumen and mixing plant, cost would work out to Rs 24 per 100 sq. ft.

Paper No. 220

# "ROAD TRANSPORT OPERATION COST ON VARIOUS TYPES OF SURFACES."

*By*

N. D. DAFTARY † AND DR. M. K. GANGULI *

### CONTENTS



### SYNOPSIS

At the instance of the Government of India, the Indian Roads Congress appointed a Committee to study (i) the cost of operation of motor vehicles on different classes of roads and (ii) the economy resulting from road improvements. As recommended by the Committee, controlled experiments were carried out by Bombay and Uttar Pradesh State Transport Undertakings according to design and specifications supplied by the Committee. This Paper gives the details of the actual experiments and their results. The results indicate in detail that there is substantial reduction in the cost of operation of motor vehicles with the improvement of road surfaces. Experience of other countries is also reviewed in this Paper.

† Deputy Chief Engineer, Irrigation, Bombay.

* Ministry of Transport & Communications, Deptt. of Transport, Roads Wing, New Delhi.

## 1. INTRODUCTION

1.1. The total cost of any highway improvement is the cost to improve and maintain the highway and to operate all the vehicles thereon. Thus the planning and designing of a highway, particularly the selection of the route and the choice between alternatives, can best be done by calculating the costs of vehicle—operation, maintenance, etc., in addition to calculating initial costs. So far as economy of road construction and maintenance is concerned, fairly adequate information is readily available but the same cannot be said about road vehicle operations. Therefore, in order to examine the extent to which upgrading of surfaces brings down the operation cost of transport, a committee of experts was appointed by the Indian Roads Congress according to the recommendations of the Transport Advisory Council, who discussed the subject in their meeting held in January 1951. It was felt that intensive effort is necessary for working out accurately the statistics of actual operation on the several types of surfaces from the major operators in the country so as to arrive at the exact quantum of difference in each case.

1.2. Statistics of motor vehicle operation as usually compiled and made available by these operators cannot throw full light on the subject under consideration. It was, therefore, decided to collect all relevant information from the motor transport operators (including State Transport Undertakings) by framing a suitable questionnaire. In response to the questionnaire issued, statistical data in some form or the other were received from 10 private operators and 4 public services. All the data could not, however, be used because some of them, specially from private operators, were incomplete and lacking in the requisite details. In particular, no private operator could supply any data throwing light on the difference in the cost of operation between surfaces of roads. As regards public services, the data received from them were not in a form to meet the requirements. Two of the services were city operated and their experience accordingly could not be used for any other service and, moreover, they were running on the same road surface, namely, black-top but with different types of vehicles. In the remaining cases also either the requisite details in regard to road surfaces were not available or the statistics were not compiled on a uniform basis.

1.3. Reviewing these results, it was concluded that reliable and convincing results could not be brought about by collecting data in the form of a questionnaire from the operators and it was felt the only satisfactory course would be to carry out controlled experiments to throw light on the problems required to be studied. In view of the difficult nature of the problem, it was clear that an experiment would have to last for sufficiently long time and only after, say, three years some precise conclusion could be arrived at. In pursuance of this decision, the possibility of designing a scientific experiment on modern statistical lines was thoroughly examined and a technical Paper[1] designing a "factorial" experiment was prepared.



1.4. The U.P. Roadways and the Bombay State Road Transport Corporation started the experiment in December 1955 and March 1956 respectively. The State Transport Undertakings of Assam and Himachal Pradesh are also to make a beginning in this respect for collecting information regarding cost of road transport with the improvement of road surfaces in the hilly regions. This Paper mainly discusses the results of the Bombay and U.P. experiments along with the findings of such experiments carried out in various countries.

## 2. MAKE-UP OF THE OPERATING COST

2.1. The total cost of operation of a vehicle consists of two parts, 'fixed costs' and 'variable costs'. The fixed costs are independent of the mileage driven and include taxes, insurance, garaging, cost of supervision and operator's wages. Depreciation of the vehicle may be either fixed or variable, depending upon the conditions of use; for example, in the case of private cars which run comparatively few miles during the course of the year, the life of the vehicle may be taken as a constant, in which event, depreciation costs become fixed. In the case of commercial vehicles, the depreciation is variable and depends on conditions of use or mileage driven.

2.2. Variable costs vary with the distance travelled and include such items as cost of fuel, oil, tyres and tubes, spare parts and maintenance. The cost of major overhauls and repairs can be treated as maintenance cost.

2.3. The total cost of operation of the vehicle in the course of a year can be converted to a vehicle mile basis in the case of passenger vehicles and to a ton-mile basis in the case of goods vechicles. Both fixed and variable costs are represented in this vehicle mile or ton-mile rate.

2.4. Variable costs are affected not only by road surface but also by other road features such as the width of roadway and its traffic lanes, the curvature and gradient of the road, etc. The latter items are special characteristics of individual routes and no attempt is, therefore, made to assess their effect on operation costs.

### 3. EFFECT OF IMPROVEMENT OF ROAD SURFACE ON OPERATING COST

3.1. An improvement in the surface of road has a specific effect indirectly lowering the cost under the several heads such as cost of fuel and oil consumed, wear and tear of tyres, consumption of spare parts, etc. Apart from this, an improved surface permits of higher speeds and this provides indirect saving in cost on account of greater mileage and consequent lesser incidence of fixed costs per vehicle mile, longer service life of the vehicle in general leading to lesser depreciation per vehicle mile, etc.

3.2. Improvement of road surface also provides intangible benefits not expressible in terms of saving in transportation cost and not easily evaluated in cash terms. These are:

(a) providing lines of communication to all parts of the producing areas and in shorter time;

(b) providing direct hauling service to all parts of the producing areas;

(c) making rural life more attractive and tending to stabilise an optimum distribution of rural and city population;

(d) increasing the social and recreational possibilities of both city and rural residence;

(e) increasing the flexibility and strength of the general transportation system of the country in times of war and emergency transportation in case rail road systems are held up by strikes or otherwise. This has been amply demonstrated during the second world war, more especially in respect of lack of road communications;

(f) raising rural land values;

(g) raising property value in both rural and urban areas;

(h) increasing the range of marketing in respect to the distance and favourable selling terms;

(i) promoting suburban residence of city dwellers and reducing the necessity for congested city living conditions;



(j) providing greater travelling comfort;

(k) reducing accidents on highways;

(l) other advantages commonly associated with modern civilisation, including such things as efficient fire and police protection, rapid postal deliveries, improved access to educational, recreational, social and health facilities, decreased cost of commodities, etc.

3.3. These intangible benifits are mentioned here for the sake of completeness, but detailed analysis has been restricted to such items of variable operating cost as can be directly expressed in monetary terms.

### 4. EXPERIENCE OF OTHER COUNTRIES

4.1. Elaborate studies on this specific problem of cost of road transport operation have been couducted in western countries, particularly in U.S.A., extending over several years and the savings in operation cost due to improvements in road surfaces have been worked out individually under each item.

4.2. The following Table[2] gives the results of an elaborate series of experiments carried out for more than thirty years in the Engineering Experiment Station at Iowa State College, U.S.A. The figures are given in cents per vehicle mile:

TABLE 1

| | Type of road surface | | |
|---|---|---|---|
| | Earth | Gravel | Pavement |
| **Variable Costs—** | | | |
| Petrol | 1.35 | 1.40 | 1.22 |
| Oil | 0.21 | 0.15 | 0.11 |
| Tyres | 0.33 | 0.44 | 0.27 |
| Maintenance | 1.01 | 0.56 | 0.10 |
| Fixed costs (based on 8,000 miles of running) | 3.07 | 3.07 | 3.07 |
| Total operating costs | 5.97 | 5.62 | 4.77 |

4.3. If, for instance, we take the 'pavement' surface as a standard for comparison, the increased cost for the other surfaces under the various heads is as follows:

TABLE 2

| | Increase in cost | |
|---|---|---|
| | Earth per cent | Gravel per cent |
| Petrol | 10.5 | 14.6 |
| Oil | 91.0 | 36.4 |
| Tyres | 22.1 | 62.8 |
| Maintenance | 910.0 | 460.0 |

4.4. Other experiments in the U.S.A. showed that, as between gravel and Portland cement concrete, for the same speed, there was a difference of 2 miles per gallon, the speed on concrete being about 40 miles and consumption one gallon for 18 miles. This means that, on gravel, compared with cement concrete, the increase in cost is $12\frac{1}{2}$ per cent.

4.5. For an assumed annual running of 14,560 miles, the total cost of operation was 0.51 cent per mile higher on gravel than on concrete roads[3]. The total corresponding cost of operation on concrete was 3.24 cents per mile and thus the gravel roads showed an increase of 15.7 per cent of total cost as against concrete roads. This conclusion compares reasonably with the Iowa results given above.

4.6. A co-operative project[4] produced results on operating costs for four identical new vehicles, two being operated on gravel and two on concrete. During the course of 260,000 miles of running, the average petrol costs on gravel were found to be ten per cent higher than on concrete, oil costs 40 per cent higher, tyre costs 100 per cent higher and repairs 150 per cent higher. Tyre punctures on gravel, compared to concrete, were in the ratio of 49 : 1 while cylinder-bore wear was twice as great, and connecting rod bearing wear almost four times as great.

4.7. According to an estimate made by Agg.[5] it would be on the side of conservatism to assume that the change from one type of road surface to the next higher type saves the automobile about one eighth of the operating cost of the lower type. This amounts to 0.8 cent per mile of travel for the average automobile on the basis of 1931 operating costs. The saving in operation cost of commercial vehicles on paved roads was estimated at 5 cents per vehicle per mile of travel.



4.8. According to the estimate of the U.S. Joint Committee on Economic Report [6], operating costs on earth roads would be approximately 2 cents per mile higher than on gravel, and 2 cents per mile higher on gravel than on paved surfaces.

4.9. In order to appreciate the magnitude of these savings relative to total operating costs, it is necessary to know also the total operating cost of each type of vehicle. The latter doubtless varies depending upon the mileage run, rate of taxes, etc., but for an overall picture the figures of total operating costs of vehicles on different high-type surfaces, given by Lawrence Lawton [7] stand as follows:

TABLE 3

| | Operating costs, cents per mile | | | |
|---|---|---|---|---|
| | Passenger cars | Light trucks | Heavy trucks | Tractor trailers |
| Congested business streets | 5.6 | 8.7 | 13.3 | 21.2 |
| Through city street | 4.5 | 6.7 | 10.0 | 16.1 |
| Semi-expressway | 3.7 | 5.7 | 8.5 | 14.0 |
| Expressway | 3.3 | 5.0 | 7.7 | 12.7 |

4.10. An estimate made in the U. K. indicated that the annual saving of the British Road Transport would be as much as Rs 125 crores if a programme of road construction and improvement aggregating to Rs 720 crores was undertaken. In addition to substantial reduction in operation and maintenance cost, the life of the vehicles is increased and there is reduction in evaluation losses. The reduction in evaluation losses was estimated at 40 per cent in Belgium recently [8]. With motor traffic an increase in mileage per gallon of fuel is possible with adequately surfaced roads and the resultant annual saving was estimated in a small country like Belgium at Rs 13 crores,

### 5. INDIAN EXPERIMENTS

5.1. In order to bring to light the effect of the condition and type of road surfaces on the cost of operation of motor vehicles, the following conditions were imposed on the Indian experiments proposed to be conducted by the various State Transport Undertakings :

(i) The experiment is to be carried out for a period of 3 to 4 full years at least. The period may be longer but on no account should it be less.

(ii) Each undertaking should choose 6 routes all on similar terrain. All the 6 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. The 6 routes chosen should have the following surfaces and conditions of surfaces :

(1) cement concrete, good and average,

(2) black-topped, good and average,

(3) water-bound macadam, good and average.

A good road means one which is in very good condition throughout, allowing a speed of 30 miles or more for the light motor vehicles. If portions are bad on account of repairs or otherwise, such lengths should not normally exceed 10 per cent of the total length.

Average condition means that the road is in a deteriorated condition and cannot allow a speed of more than 20 miles throughout its length. It would be desirable to ensure that no changes in the surface condition and type are made which might vitiate the experiment. The road should be maintained in the same standard of surface and condition.

(iii) On each route at least one vehicle should be earmarked for the experiment. All the vehicles for the experiment should be of the same make and type and year model. If only one vehicle is put on a particular road it must be new*. If more than one vehicle is put the other must be an old one @.

(iv) All the vehicles for the experiment should be fitted with new tyres of the same size and make at the beginning of the experiment. If tyres of two makes are used then half of the tyres must be of one make and half of the other. The same make of tyres on each should be fitted later on one side of the vehicles (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 in all cases.

* A vehicle being defined as new if it does not run more than 20,000 miles in all and it has not undergone any major item of repair.

@ This being defined as one which has run more than 60,000 miles and less than 100,000 miles.



(v) The spare tyres may 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 some other reason the spare tyre has to be fitted then it should be replaced by the experimental tyre as early as possible and a note should be kept recording the mileage run in the meantime so that necessary corrections may be applied.

(vi) The milometer should be in good working condition during the whole course of the experiment.

(vii) Tyres for all the vehicles should be properly inflated and maintained at the same pressure. If a difference of pressure between the front and the rear wheels is required then this difference is to be maintained in all cases.

(viii) Vehicle defects which might affect tyre life should be remedied quickly.

(ix) A tyre should be treated as worn out when the treads on it are not visible. The milages for which each tyre runs before it becomes unserviceable, i.e., when it is worn out or it bursts or receives a cut which cannot be repaired, should be recorded.

(x) In unavoidable circumstances a particular vehicle may be replaced by another of the same make, age group and condition. In such cases, if unavoidable, care should be taken to see that there is no difference in the specifications of the engine carburettor, etc., and also the tyres of the old vehicles should be fitted to the new one so that the experiment in regard to the tyre may be continued. In case a vehicle is only temporarily out of service for repairs, etc., and a substitute vehicle is put on the road in its place then the performances by the latter should not be taken into account for the experiment since the original vehicle would be deemed to be still in service but running only zero mile during the temporary period of suspension.

(xi) The same make of oil and lubricants should be used throughout in all the vehicles. Their grade may, however, be changed seasonally. But this should be done at almost the same time. Abnormal engine oil leakage should be attended to carefully.

(xii) The vehicle engines should be kept in good condition by resorting to periodical check-ups.

(xiii) The issue and maintenance programme of spare parts should ensure the vehicle maintenance in a satisfactory condition. All necessary repairs should be carried out in time.

(xiv) Drivers for the experimental vehicles should be good and experienced and they should follow correct driving methods. They should maintain safe maximum speeds. There should be a rotation of drivers in order to eliminate the drivers' effect so far as the experiment is concerned.

## 6. DETAILS REGARDING THE ACTUAL EXPERIMENT

6.1. The imposed controls enumerated above, however, had to be modified in the light of useful comments received from transport operators and experts and also in the light of experience gained by the Transport Undertakings in the course of the survey from time to time. As a result of this, it was felt imperative that

(i) The scope of the experiment should be extended to earth roads also.

(ii) The length of the routes chosen should not be limited to a range of 90 to 150 miles but may be much less, if necessary. This was particularly important in the case of concrete surfaces of which long stretches are not readily available.

(iii) The effect of road surfaces on the depreciation of the vehicles should also be taken into account. This would, however, take a long time to study from the experiment but necessary data be made available by the Bombay State Road Transport Corporation from their records.

(iv) The conditions of roads should be defined with reference to the transport vehicles themselves and not with reference to private cars. The dividing line would be at a speed of 30 m.p.h. A surface permitting, as far as possible, a continuous speed of 30 m.p.h. would be treated as 'good' and one permitting only a lower speed as 'not good'.

6.2. The Uttar Pradesh State Transport Undertaking started the experiment in December 1955 and the Bombay State Road Transport Corporation started the experiment in March 1956. The U.P. State Transport Undertaking have put 7 new



Leyland 46 seater buses on six routes, viz., 1 concrete, 4 black top and 1 W.B.M. of good surfaces while the Bombay State Road Transport Corporation has put 4 new vehicles Kew Dodge 40 seaters on 4 separate routes having surface of concrete 'good', black top 'good' and W.B.M. 'good and bad'. The Undertakings have been collecting daily data of mileage travelled, consumption of fuel, lubricants and spare parts and tyres. In all, 11 vehicles have been put on the roads for the experiment but the number of independent combinations of the various conditions available in the State is somewhat limited. All the vehicles used in the experiment are new, according to the definition having run less than 20,000 miles at the commencement of the experiment. The actual mileages run by these vehicles at the commencement of the experiment are as follows:—

| | |
|---|---|
| U.P. | 4412,4122,3333,8700,6186,2000 |
| Bombay | 234,203,486 and 349. |

The U.P. vehicles are all Leyland, of 203 in. W.B. and the Bombay vehicles are Kew Dodge of 190 in. W.B. All the vehicles are run on diesel and consequently the experimental results would rest on diesel vehicles only.

## 7. RESULTS OF THE EXPERIMENT

7.1. With a view to scientifically examining the question of savings in vehicle operation, cost of various types and conditions of surfaces, the two State Transport Undertakings in U.P. and Bombay collected daily data of mileages travelled, consumption of fuel, lubricants, spare parts, etc., in the form of six statements eliciting the following information:

Statement 'A': Particulars of roads on which experiments are being conducted.
Statement 'B': Particulars of vehicles.
Statement 'C': Spare parts consumption statement.
Statement 'D': Daily fuel consumption statement.
Statement 'E': Daily lubricant consumption statement.
Statement 'F': Tyre consumption statement.

7.2. Copies of these statements are given in Appendix 1.

## 8. FUEL CONSUMPTION

8.1. The following Table gives the average miles run per gallon of fuel by the vehicles on the different roads in Bombay for the periods for which data are now available, Table 4. The overall average miles run per gallon of fuel and the fuel cost in pies per vehicle mile are also shown for comparative study.

TABLE 4

**Monthly fuel consumption—Bombay**

(miles per gallon)

| Month | C. C. good | B. T. good | W. B. M. good | W. B. M. bad |
|---|---|---|---|---|
| 1956 March | 17.16 | 16.41 | — | 13.65 |
| ,, April | 18.20 | 16.41 | — | 13.40 |
| ,, May | 17.07 | 15.80 | — | 13.40 |
| ,, June | 16.50 | 15.83 | 16.20 | 13.24 |
| ,, July | 16.04 | 15.51 | 15.40 | 12.63 |
| ,, Aug. | 15.80 | 16.30 | 15.80 | 12.69 |
| ,, Sep. | 16.48 | 15.70 | 15.04 | 14.18 |
| ,, Oct. | 17.35 | 15.80 | 14.90 | 13.05 |
| ,, Nov. | 17.30 | 14.94 | 13.40 | 12.06 |
| ,, Dec. | 17.57 | 14.41 | 13.52 | 13.60 |
| 1957 Jan. | 18.90 | 15.20 | 13.60 | 12.20 |
| ,, Feb. | 17.80 | 14.20 | 13.00 | 12.80 |
| ,, March | 18.40 | 14.50 | 13.90 | 12.50 |
| ,, April | 18.20 | 14.10 | 14.10 | 12.30 |
| ,, May | 16.60 | 12.80 | 15.30 | 11.40 |
| ,, June | 17.30 | 12.50 | 14.20 | 11.10 |
| ,, July | 15.43 | 13.96 | 13.57 | — |
| ,, Aug. | 16.51 | 12.06 | 15.18 | 12.50 |
| ,, Sep. | 16.22 | 16.28 | 13.04 | 12.80 |
| ,, Oct. | 15.11 | 15.30 | 13.80 | 13.10 |
| ,, Nov. | 13.80 | 14.90 | 14.10 | 15.20 |
| ,, Dec. | 15.20 | 15.30 | 14.40 | 14.40 |
| 1958 Jan. | 14.70 | 15.40 | 13.60 | 13.80 |
| ,, Feb. | 15.00 | 15.00 | 14.30 | 13.90 |
| ,, March | 16.00 | 13.14 | 12.70 | 11.78 |
| ,, April | 15.52 | 11.40 | 11.94 | 11.74 |
| ,, May | 14.60 | 13.21 | 10.90 | 12.70 |
| ,, June | 14.30 | 13.75 | 11.67 | 11.40 |
| Average in mile per gallon | 16.37 | 14.57 | 13.94 | 12.80 |
| Cost in pies per vehicle mile | 17.40 | 19.76 | 20.68 | 22.60 |

N.B. The cost figures given in the last row of the Table are based on a common price of fuel of Rs 1.50 per gallon.



8.2. It is clear from the above Table that the change of road surface from bad water-bound macadam to good cement concrete reduces the fuel cost per gallon from 22.6 pies to 17.4 pies which means a saving of fuel consumption to the extent of about 25 per cent. If we restrict the comparison between water-bound macadam surfaces, good and bad of the Bombay State, it is found that a good conditioned road would save about 10 per cent of the cost incurred on bad conditioned roads in respect of fuel consumption. The results, however, indicate that good concrete sufaces influence the fuel consumption to a significant degree as compared to good black top surfaces. Amongst the good black top and good water-bound macadam surface, however, the difference is not so well marked so far as fuel consumption is concerned.

8.3. The data regarding fuel consumption in Bombay experiment also reveal the extent of increase in the consumption of fuel with the increase in the age of the vehicle. In the course of about two years of experiment, it is seen that in almost all the routes of Bombay, the mileage per gallon of fuel consumption has come down by about 2 to 3 miles. A detailed study has been carried out by taking monthly average figures giving the number of miles travelled per gallon of fuel for each month in natural order and finding out the correlation coefficients between these figures and the natural number 1,2,3 etc. indicating the order of the month. In all the cases, the correlation coefficient come out as negative indicating that the mileage travelled per gallon of fuel consumption decreases with time and since the values of these coefficients were found to be statistically significant, it can be concluded that even during this short period of experiment, the age of the vehicle has got an appreciable influence on the consumption of fuel.

8.4. Out of the seven routes chosen for the experiment in U.P., one route namely, Gorakhpur-Maharajganj having surface conditioned as kankar has undergone substantial improvements during the course of the experiment. The data for this route have, therefore, been kept separate and have not been given in the present analysis. Table 5 gives the average mileage run per gallon by the vehicles in different months on different roads in U P. The overall average miles run per gallon of fuel and the cost in pies per vehicle mile have also been indicated at the bottom of the Statement.

8.5. The results of the experiment in U.P. compare favourably with those in Bombay. For instance, superiority of the concrete surface in influencing fuel consumption comes out clearly in the results of U.P. experiment also. As regards fuel

TABLE 5

**Monthly fuel consumption—Uttar Pradesh**

(miles per gallon).

| Month | C.C. good Agra-Etawah | Blacktop—good Agra-Mathura (1) | Blacktop—good Agra-Mathura (2) | Blacktop—good Allad.-Chakghat | Blacktop—good Mirzapur-Robertsganj | Blacktop—good Gorkhpur-Kasia |
|---|---|---|---|---|---|---|
| 1955 December | 18.00 | 16.40 | 16.20 | 15.20 | 14.66 | 15.41 |
| 1956 January | 18.20 | 16.30 | 16.40 | 16.40 | 16.00 | 15.30 |
| ,, February | 19.00 | 16.70 | 16.30 | 15.50 | 16.40 | 16.07 |
| ,, March | 19.70 | 17.20 | 16.80 | 16.00 | 16.40 | 17.54 |
| ,, April | 18.70 | 17.10 | 17.10 | 15.65 | 16.24 | 18.64 |
| ,, May | 18.60 | 17.00 | 16.40 | 16.00 | 16.10 | 18.27 |
| ,, June | 17.70 | 17.00 | 15.90 | 14.40 | 16.00 | 16.17 |
| ,, July | 17.10 | 16.60 | 16.20 | 14.20 | 16.00 | 17.42 |
| ,, August | 17.30 | 15.80 | 15.80 | 14.40 | 15.20 | 17.27 |
| ,, September | 17.50 | 15.80 | 15.80 | 14.40 | 15.05 | 16.82 |
| ,, October | 17.70 | 16.30 | 15.30 | 15.20 | 15.70 | 17.16 |
| ,, November | 16.80 | 16.10 | 15.90 | 15.13 | 15.40 | 17.56 |
| ,, December | 16.40 | 16.40 | 16.70 | 15.10 | 15.30 | 17.13 |
| 1957 January | 17.30 | 17.00 | 16.40 | 14.30 | 15.20 | 16.38 |
| ,, February | 17.30 | 16.70 | 16.20 | 13.80 | 15.20 | 17.63 |
| ,, March | 17.40 | 17.00 | 17.00 | 16.60 | 15.70 | 17.97 |
| ,, April | 17.70 | 16.70 | 17.00 | 16.33 | 15.90 | 17.16 |
| ,, May | 17.20 | — | 17.00 | 15.60 | 16.30 | 17.82 |
| ,, June | 16.80 | 16.90 | 16.70 | 16.10 | 15.70 | 19.21 |
| ,, July | 17.10 | 17.10 | 16.00 | 16.26 | 15.50 | 16.61 |
| ,, August | 16.90 | 17.30 | 17.30 | — | 16.16 | 17.95 |
| ,, September | 16.60 | 17.30 | 16.80 | — | 15.50 | 18.03 |
| ,, October | 16.70 | 17.30 | 16.90 | 15.60 | 16.20 | 19.64 |
| ,, November | 17.10 | 17.70 | 16.70 | 16.06 | 16.10 | 17.14 |
| ,, December | 17.60 | 18.[illegible]0 | 16.70 | 15.[illegible]0 | 15.70 | 16.77 |
| 1958 January | 18.00 | 18.20 | 16.90 | 15.35 | 15.36 | 17.80 |
| ,, February | 18.10 | 18.40 | 17.40 | 15.58 | 15.57 | 17.93 |
| ,, March | 18.20 | 18.60 | 17.90 | 15.90 | 15.70 | 18.17 |
| ,, April | 17.80 | 18.10 | 17.90 | 16.50 | 15.78 | 17.36 |
| ,, May | 17.80 | 17.70 | 17.40 | 16.06 | 16.33 | 18.46 |
| ,, June | 18.00 | 17.30 | 17.20 | 16.36 | 16.[illegible]3 | 1[illegible].01 |
| ,, July | 17.80 | 17.30 | 17.30 | 16.30 | 16.20 | 16.91 |
| ,, August | 17.00 | 16.20 | 16.20 | 15.38 | 16.31 | 16.77 |
| ,, September | 16.50 | 16.00 | 15.90 | — | 16.16 | 17.14 |
| Average in mile per gallon | 17.59 | 17.02 | 16.68 | 15.52 | 15.84 | 17.50 |
| Cost in pies p.v.m. | 16.38 | 16.91 | 1[illegible].55 | 1[illegible].55 | 18.18 | 16.45 |

*N B.* The cost figures given in the last row of the above Table are based on a common price of fuel of Rs 1.50 per gallon.



consumption on blacktop good roads there appears to be considerable variation in the performance of the vehicles on the five routes in U P. The average fuel cost in pies per vehicle mile increases from 16.5 on Gorakhpur-Kasia route to 18.6 pies on Allahabad-Chakghat route. Rigorous statistical analysis suggest that though the vehicles are running on the same type of roads, the amount of variation observed in the rate of fuel consumption cannot be ignored. This point was examined in detail and it was found that the variations were primarily due to the reasons that Mirzapur - Robertsganj road has hilly gradients in a certain portion which adversely affect the oil consumption of the vehicles. Further, the vehicles coming from Chakghat to Allahabad have to pass through congested portions of Allahabad town and have to negotiate two major bridges which are fairly long and narrow for the traffic and buses have considerable difficulty in crossing the bridges and have to do so in low gear.

8.6. While the results so far obtained prove the superiority of the high-grade surfaces, one can hardly overlook the necessity of extending the scope of the experiment by including some bad conditioned roads and hilly roads. There is only one bad road, i. e., water-bound macadam 'bad' in the experiment conducted in Bombay and in this route the load factor again is relatively much less as compared to those in other routes. The average load factor on the water-bound macadam 'bad' route is about 60 per cent as compared to 70 - 75 per cent on other routes. Thus the findings given above though may be sufficiently helpful for finding out the cost of operation in different types of roads, namely, concrete, blacktop and water-bound macadam but necessarily have limited validity in respect of condition of roads for which very little data are available. This matter has been taken up by the expert Committee and the scope of the experiment is now being extended to hilly regions. Nevertheless, it is necessary to include some bad or at least fair type of modern surface roads for wider inference.

## 9. LUBRICANT CONSUMPTION

9.1. The variations in the lubricant oil consumption of the the vehicle plying on different routes in Bombay and U. P. are relatively small as shown in Tables 6 and 7.

The costs in the consumption of lubricant oil are not usually reduced with the improvement in type or condition of the surface. In fact, it is extremely difficult to estimate the small difference in the consumption of lubricant oil due to the type

TABLE 6

**Lubricant consumption—Bombay**

(Cost in pies per vehicle mile)

| Period | C. C. good | B. T. good | W.B.M. good | W.B.M. bad |
|---|---|---|---|---|
| March '56 to August '56 (6 months) | 2.02 | 1.87 | 1.89 | 1.86 |
| Sept. '56 to Feby. '57 (6 months) | 2.12 | 1.95 | 1.67 | 1.46 |
| March '57 to August '57 (6 months) | 2.19 | 2.26 | 2.04 | 1.70 |
| Sept. '57 to Feby. '58 (6 months) | 2.15 | 1.84 | 1.96 | 2.11 |
| March '58 to June '58 (4 months) | 2.11 | 1.90 | 2.40 | 2.84 |
| Overall March '56 to June '58 (28 months) | 2.11 | 1.96 | 1.94 | 1.98 |

and condition of the route from the experimental data. The expert advice of the oil companies of Bombay was also sought in this connection on the subject. The general views were that there are other factors like types of oil filters and crank-case ventilation, etc., which are more responsible for this than either the condition or the type of the surface.

9.2. It is, however, noted that on the average, in respect of lubricant oil consumption, Bombay vehicles are costing double than those of U. P. It was found that the vehicles in Bombay are having more frequent oil changes than the vehicles in U. P. In Bombay the lubricant consumption on the average in a month comes to about seven gallons for a monthly mileage of 3,300. The vehicles (in Bombay) are having oil change on the



TABLE 7

**Lubricant consumption—Uttar Pradesh**

(Cost in pies per vehicle mile)

| | ............C. C. Good | | | Blacktop............ | | |
|---|---|---|---|---|---|---|
| Period | Agra-Etawah | Agra-Mathura (1) | Agra-Mathura (2) | Allaha-bad-Chakghat | Mirzapur-Roberts-ganj | Gorakh-pur-Kasia |
| Dec. '55 to May '56 (6 months) | 0.91 | 1.02 | 1.01 | 1.15 | 1.62 | 0.95 |
| June '56 to Nov. '56 (6 months) | 1.03 | 1.08 | 1.06 | 0.94 | 1.17 | 0.97 |
| Dec. '56 to May '57 (6 months) | 0.76 | 1.22 | 0.99 | 1.02 | 1.18 | 1.20 |
| June '57 to Nov. '57 (6 months) | 1.03 | 1.40 | 0.86 | 0.84 | 1.05 | 1.47 |
| Dec. '57 to May '58 (6 months) | 1.28 | 1.23 | 1.56 | 1.00 | 1.67 | 1.22 |
| June '58 to Sept. '58 (4 months) | 1.63 | 1.38 | 1.59 | 1.32 | 1.36 | 0.83 |
| Overall Dec. '55 to Sept. '58 (34 months) | 1.08 | 1.21 | 1.15 | 1.04 | 1.34 | 1.11 |

*N.B.* The cost figures given in the last row of the above Table are based on a common price of Rs 4.50 per gallon of lubricant.

average twice in a month to the tune of 2.5 gallons each time and the top-up per month is about 2 gallons. The comparable figures in case of U.P. eome to 3 gallons for oil change which is generally done once in a month and 1.75 gallons top-up. It, therefore, follows that while the top-up in the two cases does not vary much, the lubricant oil change is about double in the case of Bombay as compared to U. P.

## 10. TYRE CONSUMPTION

10.1. Table 8 shows the life of tyres on different routes as revealed by the experiments conducted in Bombay and U.P.

TABLE 8

**Tyre consumption - life of tyre in miles, Bombay**

| | C. C. good | B. T. good | W. B. M. good |
|---|---|---|---|
| | 47,340.85 | 22,169.5 | 15,776.9 |
| | 49,090.15 | 22,169.5 | 15,776.9 |
| | 49,090.15 | 25,432.5 | 15,776.9 |
| | 49,090,15 | 25,432.5 | 9,613.5 |
| | 49,090.15 | 25,432.5 | 16,513.9 |
| | 49,090.15 | 25,432.5 | 19,832.3 |
| | | 22,169.5 | 13,694.0 |
| | | | 20,089.5 |
| | | | 10,700.6 |
| Average life of a tyre (in mile) | 48,798.60 | 24,034.1 | 15,308.3 |
| Cost in pies per tyre mile | 1.77 | 3.59 | 5.64 |
| Cost in pies per vehicle mile | 10.62 | 21.54 | 33.84 |

10.2. Taking the vehicles in Bombay, it is found that the wear and tear of tyres is less with the improvement of the type of surface. In fact, the figures of the Bombay experiment bring out that the life of the tyres used on concrete 'good' road is treble that used on water-bound macadam 'good' roads. The results of the U.P. and Bombay experiment confirm that the wear and tear of tyre is much less on concrete 'good' roads in India. The average life of tyre on concrete 'good' road as found from U.P. data, compares very favourably with that of Bombay, the figures being 50 and 49 thousand miles respectively. The life of tyre on blacktop 'good' road in U. P. is varying between 30,000 to 40,000 miles and is consistently higher than that on



TABLE 9

**Tyre consumption - life of tyre in miles, Uttar Pradesh**

| | C.C. good Agra-Etawah | Black top | | | | |
|---|---|---|---|---|---|---|
| | | Agra-Mathura (1) | Agra-Mathura (2) | Alld-Chakghat | Mirzapur-Roberts-ganj | Gorakh-pur-Kasia |
| | 47,893 | 32,472 | 35,516 | 50,499 | 32,368 | 17,843 |
| | 51,838 | 35,210 | 32,306 | 41,972 | 35,320 | 36,080 |
| | 52,486 | 34,830 | 38,122 | 42,719 | 36,038 | 35,109 |
| | 52,490 | | 26,078 | 29,410 | 28,419 | 36,249 |
| | 52,490 | 20,494 | 22,416 | 41,448 | 40,969 | 42.311 |
| | 50,931 | 34,681 | 33,255 | 43,606 | 40,666 | 40,068 |
| | 49,532 | 24,837 | 31,407 | | 34,807 | 32,280 |
| | 42,832 | 24,364 | 22,777 | | 24,656 | |
| | | 38,617 | 38,147 | | 34,216 | |
| | | 43,159 | 41,089 | | 34,627 | |
| | | 44,093 | 39,063 | | | |
| | | 42,031 | 35.258 | | | |
| | | 43,117 | 39,526 | | | |
| | | 6,499 | 36,239 | | | |
| | | 34,881 | | | | |
| Average life of a tyre (in mile) | 50,061.5 | 32,806.0 | 33,657.0 | 41,609.0 | 34,208.0 | 34,277.0 |
| Cost in pies per tyre mile | 1.72 | 2.63 | 2.57 | 2.08 | 2.52 | 2.52 |
| Cost in pies per vehicle mile | 10.32 | 15.78 | 15.42 | 12.48 | 15.12 | 15.12 |

*Note*: The above figures of cost in pies per tyre mile are based on uniform price of Rs 450 per tyre.

the same type of surface in Bombay, which comes to about 24,000 miles. In terms of cost per tyre-mile or vehicle-mile the above conclusion remains unchanged. Since the load factor and other conditions of water-bound macadam 'bad' road in Bombay are not comparable with those of other routes, the data for that route could not be utilised for valid comparison.

## 11. SPARE PARTS CONSUMPTION

11.1. The consumption of spare parts as revealed in the experiment is very much influenced by the type of surfaces. With the increase in the age of the vehicles the consumption of spare parts is also increasing considerably. In some cases the life of engine and other costly parts are lately coming into the picture. The following Table gives the consumption of spare parts on different types of routes in Bombay:

TABLE 10

**Spare parts consumption (excluding batteries and tyres)**

| Period | Cost in pies per vehicle-mile | | |
|---|---|---|---|
| | C.C. good | B.T. good | W.B.M. good |
| March '56 to August '56 (six months) | 1.90 | 1.93 | Nil |
| Septr. '56 to Feb. '57 (six months) | 1.47 | 1.72 | 3.65 |
| March '57 to August '57 (six months) | 9.65 | 1.94 | 7.56 |
| Septr. '57 to Feb. '58 (six months) | 1.00 | 6.12 | 21.31 |
| March '58 to June '58 (four months) | 0.42 | 12.81 | 42.08 |
| Overall March '56 to June '58 (28 months) | 3.01 | 4.28 | 12.42 |

Though the effect of the different types of surfaces has come out prominently in the above figures, the pattern of consumption of spare parts has, however, not yet stabilised. The experiments are, therefore, being continued for some more time so that conclusions in respect of spare part consumption and tyre-life can be made on a firm basis.

## 12. CONSOLIDATED PICTURE

To sum up, the experiments conducted by Bombay and U. P. State Transport Undertakings have clearly brought out the amount of reduction in the cost of operation of motor vehicles in different types of surfaces. A consolidated picture is given below:



TABLE II

**Variable operation cost in different surfaces**

| | Type of road surface | | |
|---|---|---|---|
| | Concrete good | Blacktop good | W.B.M. good |
| | (approximate cost in pies per vehicle-mile) | | |
| Fuel | 17.0 | 18.5 | 20.5 |
| Lubricant | 1.5 | 1.5 | 1.5 |
| Tyre | 12.0 | 15.0 | 30.0 |
| Spare parts | 3.0 | 4.0 | 12.0 |
| Total variable cost in pies per vehicle-mile | 33.5 | 39.0 | 64.0 |

The above figures of fuel and lubricant oil are based on a common price of Rs 1.5 and Rs 4.5 per gallon and of tyre Rs 450.0 per tyre. As the experiments are still in progress no attempt has been made to estimate the fixed cost and correlate the reduction in the cost of operation with the overall expenditure. With the extension of experiment in the hilly regions and on different conditions of roads it is hoped that wider inference can be made with validity in respect of reduction in the cost of operation for varying condition of roads.

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## Appendix 1

### Experiment for assessing the cost of operation of motor vehicles on different types of road surface

#### Statement-A

Month............195 .

**Particulars of routes on which experiment is being carried on**

| S. No. | Name of route | Route mileage | Type of road surface | Condition of road surface | Running time for a single journey |
|---|---|---|---|---|---|

#### Statement-B

Month............195 .

**Particulars of vehicles**

| No. | Bus No. | Make | Wheel base | Seating capacity | Whether Tachograph is fitted in the vehicle | Name of route on which the vehicle was put on road | * Date on which the vehicle was put on road | Miles covered before putting the vehicle on road | Date of taking vehicle off the road | Mileage covered during the month | Total miles covered | Remarks, if any |
|---|---|---|---|---|---|---|---|---|---|---|---|---|

* It would be the date on which the experiment was started by putting the vehicle on the route selected for the purpose.



#### Statement-C

Month............195 .

**Spare part consumption statement**

Vehicle No. Make of vehicle : Leyland

Name of route on which the vehicle was run :

| S. No. | Date of issue of the spare parts | Name of spare parts issued to the vehicle | New (N) or reconditioned (R) | Quantity | Cost of the part issued | Remarks, if any |
|---|---|---|---|---|---|---|

#### Statement-D

Month............195 .

**Daily fuel consumption statement**

Vehicle No. Make of vehicle : Leyland. Type of fuel in use : diesel

Name of route on which the vehicle was run :

| S. No. | Date | Fuel consumed in gallons | Miles covered | Average miles per gallon | Cost of fuel consumed Rs. As. Ps. | Remarks, if any |
|---|---|---|---|---|---|---|

**Statement-E**

Month............195 .

**Daily lubricant consumption statement**

Vehicle No. Make of vehicle: Leyland
Name of route on which the vehicle was run:

| S. No. | Date | Lub. consumed in gallons | Miles covered | Average miles per gallon | Cost of lubricant consumed Rs. As. Ps. | Remarks, if any |
|---|---|---|---|---|---|---|

**Statement-F**

Month............195 .

| Vehicle No.......... | S. No. | Tyre No. | Size | Position | Date of fitting |
|---|---|---|---|---|---|
| Make: Leyland<br>Name of route on which the vehicle was operated | 1 | | | | |
| | 2 | | | | |
| | 3 | | | | |
| | 4 | | | | |
| | 5 | | | | |
| | 6 | | | | |

| S. No. | Date | No. of tyre removed from vehicle | Purpose of removal | Miles covered by the tyre at the time of removal | Remarks, if any |
|---|---|---|---|---|---|

# RESEARCH SECTION

# "A STUDY OF SOME FACTORS INFLUENCING THE SWELLING PRESSURE OF CLAYS"

*By*

DR. H. L. UPPAL & R. M. PALIT*

CONTENTS



## 1. INTRODUCTION

It is a common observation that when compacted soil is allowed to come in contact with water, the moisture content and the volume of the mass increase. The phenomenon is known as swelling. It is of considerable importance in engineering construction because the swelling of soil results in the loss of shearing strength which contributes greatly to the bearing capacity of soils.

The swelling is mostly attributed to the different grain size fractions, and each particle exhibits this property to a different magnitude depending upon its size and clay mineralogy. That is why the causes of swelling for sandy soils, clayey soils, and predominantly fat clays may be different in each case. Since the swelling caused in sandy or silty soils is very small as compared to the fat clays, the word swelling is invariably used in this country for soils such as black cotton soil, or red earths. As already mentioned, all clay particles do not exhibit the same degree of swelling and, therefore, this type of phenomenon cannot be attributed to the colloidal nature only.

When clay swells, a lot of pressure is developed resulting in the damage to the structure. The swell pressure of clays varies

* Central Road Research Institute, New Delhi.

considerably from place to place depending upon the climatic environments. It has been observed[1], that black cotton soil in a certain part of Burma exerted a swell pressure of about 1 ton per sq. ft. as a result of rain effect over-night. In Texas[2], a swell pressure of about 3.2 tons per sq. ft .has been reported. In Nigeria, which abounds in montmorillonite type of soil, a swell pressure of 11.5 tons[3] per sq. ft. has been recorded.

Since the texture of soils under the foundations of buildings, roads or runways would change at frequent intervals, the swell pressure will also be of widely different nature, causing a more serious damage than if soil with a uniform texture existed all along the length. Many buildings in South Africa[4] have failed not on account of uniform high swell pressure but on account of differential swelling.

## 2. CAUSES OF SWELLING

Research work has been going on in various countries to find out the reasons for swelling of clays, and one of the reasons which has been more or less universally accepted is the swelling characteristics of the clay mineral. On the basis of structure, clays have been divided into three main groups, i.e., kaolinite, illite and montmorillonite. Out of these, the montmorillonite is the most swelling type. The outstanding feature of the montmorillonite structure is that water and other polar molecules such as certain organic molecules can enter between the unit layers causing the lattice to expand in the vertical direction. The vertical axis dimension of montmorillonite is not fixed but varies from 9.6 A° (where no polar molecules are between unit layers) to 21.4 A° (where four molecules of water are absorbed) resulting ultimately in complete separation of the individual unit. It is on account of this that montmorillonite is termed as expanding lattice clay while others are termed as fixed lattice clays. Attempts have also been made by some workers[5] to explain the causes of swelling on the basis of negative pressure created on desiccation which is neutralised when dry soil comes in contact with moisture and swells. Yet another view held by some workers [6] in India is that swelling of soils is a function of the surface area since montmorillonite clays have surface area of about 250-300 sq. cm/gm of soil as compared to kaolinite which has 8-10 sq. cm/gm the former type of clays exerts very much higher swell pressure than the latter.



## 3. TESTS FOR IDENTIFICATION OF SWELLING SOILS

A number of methods are known which can identify montmorillonite from kaolinite. Important amongst these are:

### (i) Base exchange capacity

The base exchange capacity of montmorillonite soils is about 40-100 m.e./100 gm. as compared to *illite* which has 20-40 m.e./100 gm. and kaolinite which has only 3-20 m.e./100 gm.

### (ii) Silica sesquioxide ratio

The silica sesquioxide ratio of montmorillonite clays is higher than kaolinite. The former has silica sesquioxide ratio of above 3 whereas the latter has generally below 2.

### (iii) Differential thermal analysis

The montmorillonite clay has a peculiar D.T.A. curve widely different from kaolinite.

### (iv) Electronic microscope

The electronic microscope has also been used successfully to distinguish between montmorillonite and kaolinite.

### (v) X-ray

The X-ray photograph can be used very successfully for identifying clay minerals.

### (vi) Surface area

The surface area of montmorillonite clays is almost 30-40 times that of kaolinite clays. The surface area can be determined by a number of methods such as vapour pressure, glycerol ether method, etc. It can also be approximately determined from the degree of absorption of a colouring matter.

## 4. THE LABORATORY INVESTIGATION

4.1. To study the effect of various factors which influence the swell pressure of a black cotton soil, a detailed investigation was carried out.

Black cotton soil of India has been identified as predominantly montmorillonite[7] and consequently of a swelling type.

Since the soil is scattered over a big area, it varies considerably in plasticity index and fraction coarser than 200 U. S. sieve. For purposes of this study the medium type of soil was selected, the physical characteristics of which are given below :

| Sand per cent fraction coarser than 200 and finer than 40 U.S. sieve | Clay per cent <.002 mm | Liquid limit per cent | Plasticity index | Shrinkage limit per cent | Optimum moisture content |
|---|---|---|---|---|---|
| 12 | 45 | 53.5 | 30.2 | 9.0 | 15.0 |

### 4.2. Method Adopted for the Determination of Swell Pressure

The method of determining the swell pressure in the past consisted in putting the soil in the consolidation apparatus and finding out the over-burden pressure which could stop the swelling when the sample is allowed to come in contact with free water. The method was considered to be fairly laborious and time consuming. Subsequent work on the subject carried out by various workers [8] had simplified the method and the same has been adopted in the study.

## 5. RESULTS OF INVESTIGATIONS

5.1. The following aspects of the problem have been studied in the laboratory :

(i) The effect of degree of compaction on the swell pressure.

(ii) The effect of height of the soil on swell pressure.

(iii) The effect of initial moisture content of the soil on the swell pressure on saturation.

(iv) The effect of confinement on the swell pressure.

### 5.2. Effect of Density on the Swell Pressure

It has been mentioned by various workers that swell pressure of clays depends upon void ratio but no detailed data about the exact swell pressure at various void ratios by different soils has so far been reported. This aspect of the problem was investigated by compacting the soil to dry bulk densities ranging



from 1.35 to 1.75. As it is not possible to compact black cotton soil conveniently beyond 1.75 density in the field, the studies on densities higher than this were not carried out in the laboratory. Blocks were prepared by compacting soil at optimum moisture (15 per cent) in a mould 2.5 in. dia. The height of the specimen was kept at 2.8 in. The freshly compacted blocks were allowed to be saturated by keeping the base in contact with free water. The maximum swell pressure recorded at saturation is given in Table 1 and plotted in Fig. 1. It will appear from the results that for the same height of the block, swell pressure increases

TABLE 1

**Showing the effect of increasing densities on the swell pressure of black cotton soil**

| Density in gm/c.c. | Swelling pressure in lb/sq. in. |
|---|---|
| 1.35 | 0.5 |
| 1.40 | 0.8 |
| 1.45 | 1.6 |
| 1.50 | 2.4 |
| 1.55 | 3.5 |
| 1.60 | 5.4 |
| 1.65 | 8.0 |
| 1.70 | 10.5 |
| 1.75 | 12.4 |

with the increase in dry bulk density. It will further appear that for low densities upto 1.5 gm/c.c., the swell pressure is very small, that is, about 2.5 lb per sq. in. but as the degree of compaction is increased beyond this there is an abrupt rise in the swell pressure so much so that at 1.75 the swell pressure is about 12.5 lb per sq. in. The results have a practical bearing on the use of black cotton soil in the field as it indicates that in areas which are likely to get completely dehydrated and hydrated, it may not be advisable to compact the soil to a high density with the object of attaining increased load bearing capacity, as is generally done with alluvial soils. The relationship between swell pressure and dry bulk density has also been expressed on semi-logarithmic basis in Fig. 2. It will have the advantage that mathematical relationship can be worked out for swell pressure

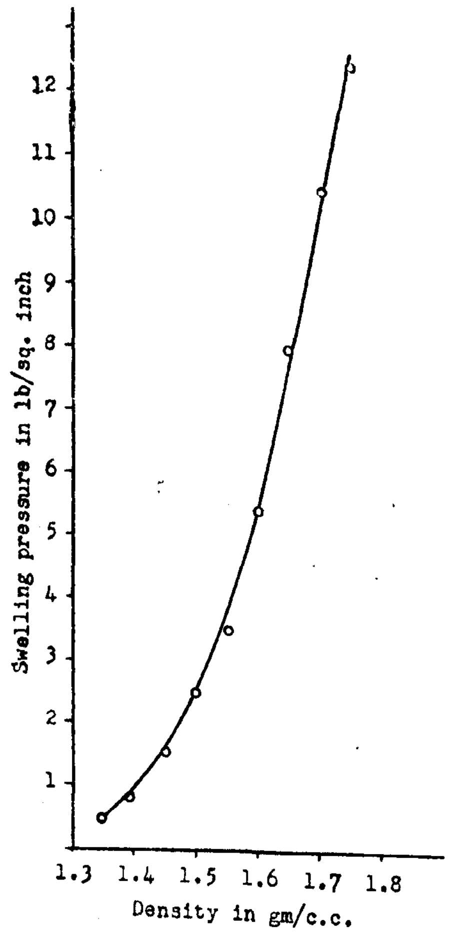


Fig. 1. Effect of dry bulk density of soil on swell pressure

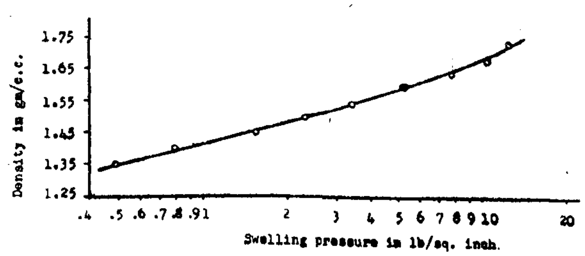


Fig. 2. Effect of dry bulk density of soil on swell pressure reproduced on semi-log basis



at various densities if values for only two densities are known. It is, however, not possible to comment in detail for want of similar data on other types of black cotton soil.

### 5.3. Effect of Height on the Swelling Pressure of Soil

To find out the effect of height on the swelling pressure of soil, the following experiment was conducted :

The soil was compacted in a mould of 2.5 in. dia. at optimum moisture content to a density of 1.6 gm/c.c. The height of the sample varied from 1.2 in. to 13 in. (3 to 33 cm.) by compacting a known weight of soil to a particular height under static load. The swelling pressure of the various samples was determined after the compacted blocks had been saturated. The results of the tests are given in Table 2 and reproduced in Fig. 3. It will be seen from the results that as the height of the block

TABLE 2

**Showing the effect of height of soil on the swelling pressure of black cotton soil**

Density of soil block = 1.6 gm/c.c.

| Height in inch | Swelling pressure in lb/ sq. in. |
|---|---|
| 1.2 | 2.1 |
| 2.8 | 4.5 |
| 4.4 | 6.2 |
| 6.0 | 8.6 |
| 8.5 | 12.5 |
| 10.5 | 16.3 |
| 12.0 | 18.5 |
| 13.2 | 20.5 |

under test increases, the swell pressure also increases, which means that swelling is proportional to the depth of the same and is a cumulative effect of expansion of each layer. The results have a far reaching effect as the mere determination of swell pressure of the various soils in the laboratory will not give

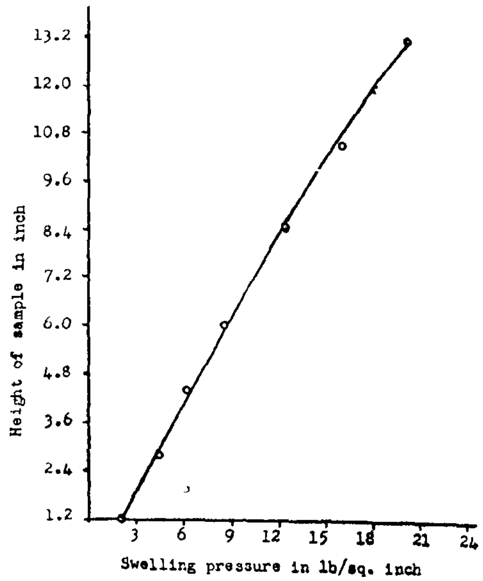


Fig. 3. **Effect of height of soil on swell pressure**

any comparative idea unless the depth to which each soil lies in the field is also known.

### 5.4. Effect of Initial Moisture Content of the Soil on the Swell Pressure at Saturation

Since swell pressure is a function of the degree of saturation, the laboratory tests are normally carried out by taking into consideration extreme limits of moisture content. The dry block is allowed to absorb moisture and swell pressure is determined when it is completely saturated. These extreme conditions of moisture are, however, not always likely to exist in actual field conditions and, therefore, the laboratory results are likely to be high and unreal. In most of the cases, the soil as it lies under the road base may not desiccate completely and also may not get saturated completely under adverse moisture conditions. The actual swell pressure will, therefore, depend upon the climatic conditions of the area and, therefore, the same soil will be exerting different swell pressures at different places. With the object of knowing the effect of particular range of moisture on the development of swell pressure, the black cotton soil under investigation was compacted in a mould as used previously. A number of blocks of different densities were made



at different moisture contents. These blocks were then allowed to get saturated and swell pressure was determined. The results for dry bulk densities of 1.6 and 1.8 are given in Table 3 and reproduced in Fig. 4. It will appear that swell pressure for initial moisture content of 3.6 and 9 per cent for this particular type of soil is the same. But beyond that it decreases as the initial moisture content of the block increases, till at saturation no

TABLE 3

**Showing the effect of initial moisture content on the swelling pressure of black cotton soil**

(a) Density 1.6 gm/c.c.

| Initial moisture content | Swelling pressure in lb/sq. in. |
|---|---|
| 2.8 | 9.9 |
| 6.5 | 9.7 |
| 9.5 | 9.0 |
| 11.4 | 7.8 |
| 15.5 | 4.8 |
| 18 | 3.5 |
| 22.5 | 0.6 |

(b) Density 1.8 gm/c.c.

| Initial moisture content | Swelling pressure in lb/sq. in. |
|---|---|
| 2.8 | 20.9 |
| 7.5 | 20.7 |
| 10.5 | 19.5 |
| 12.2 | 15.8 |
| 14.6 | 12.1 |
| 18.0 | 4.9 |
| 18.5 | 3.7 |
| 19.5 | 1.9 |

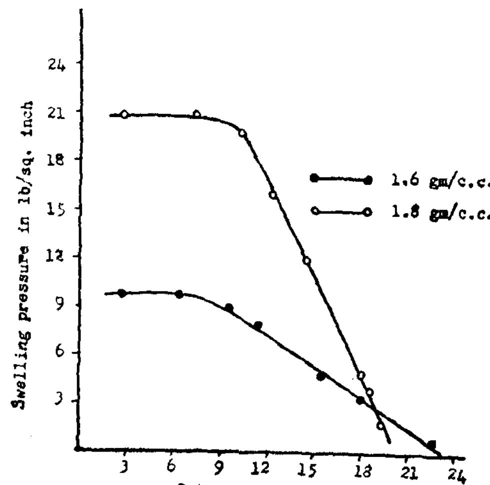


Fig. 4. **Effect of initial moisture on swell pressure of soil when allowed to saturate**

swell pressure is observed. This is true both for dry bulk densities of 1.6 and 1.8 except that the values of swell pressure at all stages of moisture are higher for higher densities.

It can be inferred from these figures that the range of moisture between which a black cotton soil under the base of a road fluctuates during the year is a governing factor in the development of swell pressure, e.g., when the moisture of the soil changes from 18 to 22.5 per cent for a density of 1.6, the swell pressure developed is only 2.9 lb per square inch, whereas if the soil at the same density had desiccated to 9.5 per cent moisture and then got saturated as a result of fluctuation of water table or other reasons, it would have developed a swell pressure of 9.4 lb per square inch. The intensity of swell pressure further increases with the rise in density, i.e., the same soil at 1.8 density when it desiccates to a moisture content of about 8 per cent and afterwards gets saturated would develop a swell pressure of 19 lb per square inch.

It will further appear that the swell pressure for all initial moisture contents up to 9 per cent in this particular soil is the same. It will not be incorrect to infer from this that swell pressure that will develop when the moisture content varies between 0 & 9 per cent for this particular soil will be almost insignificant. The shrinkage limit of this soil is also 9 per cent.



This means that so long as moisture variation of soils is within their shrinkage limits, no swell pressure will develop, and beyond that, swell pressure will depend upon the difference between a particular moisture and saturation.

### 5.5. Effect of Confinement on the Swell Pressure

The effect of confinement on swelling pressure was investigated in the laboratory by compacting black cotton soil at optimum moisture in the C. B. R. mould. A number of samples were similarly prepared and allowed to saturate under swell pressure measurement assembly. The confinement in each sample was varied by enclosing the loaded area, which consisted of 1 in. diameter plate, with annular plates varying in outer diameter from 2 in.—6 in. (6 in. being the internal diameter of the mould). Tests were carried out at three different densities of 1.65, 1.75 and 1.82 gm/cc. The results are given in Table 4 and plotted in Fig. 5. It will appear from the results that the swell pressure increases as the area of confinement increases.

TABLE 4

**Showing the effect of confinement on the swelling pressure of black cotton soil**

| Confinement in sq. in. | Swelling pressure in lb/sq. in. | | |
|---|---|---|---|
| | Density 1.82 gm/c.c. | Density 1.75 gm/c.c. | Density 1.65 gm/c.c. |
| Nil | 60 | 48 | 39 |
| 2.35 | 84 | 60 | 48 |
| 6.3 | 102 | 73 | 65 |
| 11.8 | 125 | 92 | 76 |
| 18.8 | 140 | 112 | 102 |
| 27.3 | 155 | 127 | 112 |

This is true of all densities. It further appears from the trend of the curves that the effect of confinement is practically the same on the swell pressure irrespective of the dry bulk density to which a soil is compacted, e.g., the increase in swell pressure with a confinement of 6 in. annular plate is almost 2.5 times the swell pressure at no confinement. This is true for all degrees of compaction.

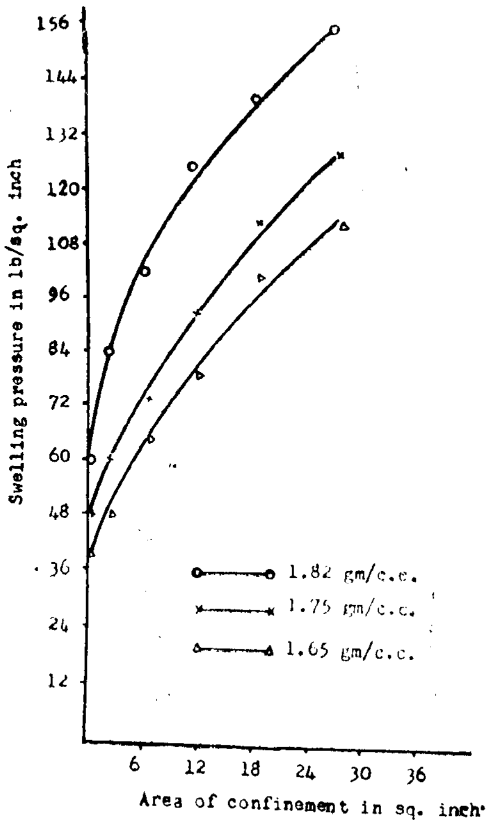


Fig. 5. Effect of confinement on swell pressure

The results have far reaching effect on the design of roads, airfields and floors of houses constructed in black cotton soil areas. Since the swell pressure increases with the area of confinement, more thickness of pavement shall have to be provided in the centre as compared to the sides. The actual difference in the thickness may have to be determined for each type of soil at natural density.

In some black cotton soil areas of Burma and South Africa, progressive heaving of floors in the centre has been observed in a number of cases. It was thought[4] to be due to the higher moisture content in the centre as compared to the sides. On actual investigation it was, however, found that the variations were not large enough to cause differential heaving. The progressive heaving can now be explained on the basis of above experiments, i.e., the centre being confined by bigger areas had resulted in the development of higher swell pressure in the subgrade as compared to the sides.



## ACKNOWLEDGMENT

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# INFORMATION SECTION

# "ULTIMATE LOAD CARRYING CAPACITY OF COMPOSITE BEAMS"

*By*

P. C. VARGHESE*

CONTENTS



SYNOPSIS

A review of work done on behaviour of steel joists encased in concrete is given and a new rapid procedure for calculation of the ultimate strength of encased standard steel joists by means of tables such as Horne's tables is proposed. Such a rapid method becomes useful for design of shear connectors for encased steel beams for bridges and other structures.

Results of actual tests made at the Civil Engineering Department, Indian Institute of Technology, Kharagpur, substantiate the proposed procedure.

## 1. INTRODUCTION

1.1. Steel frames of multi-storeyed buildings are generally encased in concrete for fire protection, architectural beauty and weather resistance. Similarly, composite beams with shear connectors are extensively used at present for economical design of bridge floors. Such type of construction has also application in reservoirs, strengthening of bridges, etc., the steel joist often being supplemented by the addition of reinforcing rods.

* Indian Institute of Technology, Kharagpur.

1.2. The encasement not only adds to the stiffness of the steel joist against premature failure by buckling in compression, but it also assists in strengthening the steel beam, the strength of a composite beam being much more than that of the plain steel beam.

1.3. Even though recent codes of practice for buildings have recognised the stiffening action of encasement by allowing a greater compressive stress to be used in the design, the contribution of concrete to direct strengthening of the beam is still not recognised in these codes. Detailed recommendations have been laid down by AASHO for design of composite beams for bridges. These state that composite beams should always be used only with proper mechanical shear connectors.

1.4. These shear connectors for composite beams are specified to be designed so that full composite action is to be retained at all levels of loading up to ultimate loads[1]. Moreover, the true over-load capacity of these beams for accidental over-loads can be computed only from a knowledge of their ultimate load carrying capacity. Hence, a quick procedure to evaluate the ultimate strength of these beams is desirable. This Paper gives a brief review of investigations on encased beams and presents a procedure for quickly computing the ultimate moment of resistance of a given composite beam made of any standard steel joist and concrete cover or slab with the use of tables like that published by Horne for "plastic moment of resistance of steel joists under compressive loads".[2]

## 2. BEHAVIOUR OF COMPOSITE BEAMS

2.1. If a composite beam is loaded to failure, three separate stages may be noticed in its behaviour.

2.2. In the first stage the beam is elastic. The distribution of strain and stress are both elastic throughout the beam. The end of this stage is reached when maximum stress in the beam reaches the yield point value.

2.3. During the second stage of loading, yielding of beam spreads from the flanges to the web and the stress in the concrete enters plastic range. (This is possible as the strain at the commencement of the yield of steel is of the order of 0.0015 and that of concrete is of the order of 0.003, well above the yield strain of steel). The load increases throughout the stage of loading and deformation increases at a rapid rate. At the end of this stage, concrete crushes and fails. The corresponding maximum moment is called the ultimate carrying capacity of the beam.



2.4. After crushing of concrete, the beam can still carry a load lower than the ultimate, in equilibrium. This last stage can be looked upon as the third stage of loading.

## 3. HISTORICAL REVIEW

3.1. The first series of tests on encased beams were made in U.S.A. in 1928 to give guidance for the strengthening of some badly corroded bridge girders. Later, Japanese investigators examined the effects of encasement in resisting earthquake shocks.

3.2. In England, results of short term loadings and sustained loadings were published by Batho, Lash and Kirkham in 1939[3].

3.3. Theoretical investigation and laboratory tests were made on shear connectors (1949) in the University of Illinois under Seiss and rules for the design of these shear connectors were evolved[4]. Approximate methods of ultimate analysis were also presented.

3.4. In 1951 Jones[5] carried out a series of tests on encased steel joists with small concrete cover of 1 in. to 3 in. as is done in fire proofing. He found that the contribution towards increase in ultimate strength by concrete was large.

## 4. SUMMARY OF FINDINGS BY THE PREVIOUS INVESTIGATIONS

### 4.1. Experiments by Batho and Associates

4.1.1. Batho's experiments (1938) were mostly confined to steel joists (4 in. × $1\frac{3}{4}$ in. at 5 lb) encased in concrete. ($\frac{3}{4}$ in. down aggregate and water cement ratios of 0.59 and 0.74 giving 5500 lb per sq. in. and 3100 lb per sq. in. concrete were used.) Both short term loading to destruction on 15 specimens and sustained loading at approximate working load on 13 specimens were made.

4.1.2. The beams were provided with excessive cover without any special form of shear connectors. Hence, most of the beams in the short term loading tests failed by bond slip.

4.1.3. As a result of these tests, the following conclusions were drawn :

(a) If bond is sufficient to ensure composite action, the stresses in the elastic stage in composite beams may be estimated by the methods used in reinforced concrete.

(b) The joist in the composite beam is in general subjected to axial and flexural stresses. The flexural resistance of the joist is much greater than that of ordinary reinforcement and may be a considerable proportion of total resistance to bending.

The flexural and axial stresses from experiments compared with theoretical values.

The concrete encasement is also under axial and flexural stresses, the axial force being equal and opposite to axial force in the steel.

(c) Large fluctuations and sudden change of stress occurred in the beams tested under sustained loads out of doors due to temperature and moisture changes. These showed the importance of efficient anchorage. Unless the bond stress is small (50 to 60 lb per sq. in.) mechanical devices should be incorporated for proper anchorage.

(d) The bond between the vertical plate and concrete is usually good but it was found weak on the underside of horizontal plate.

### 4.2. Experiments by Newmark and Associates 1936-54

A large number of pull-out tests, model tests and full scale tests were made at the University of Illinois to determine the nature of composite action, rules for design of shear connectors and to determine the ultimate strength at failure. The following observations were made:

(a) The analysis of composite steel and concrete beam by the usual transformed area method is satisfactory so long as both the materials remain elastic and there is no slip between these materials. Transformed area method is not applicable (i) when there is slip and the interaction between steel and concrete is incomplete, (ii) for estimating the ultimate load carrying capacity.

(b) Bond between concrete and steel surfaces may provide an effective shear connection so long as it is present. Results of both laboratory and field experiments have shown that bond cannot be relied upon as a permanent shear connector. Also, as the deflection at ultimate load is appreciable, the concrete tends to separate from the steel surface so that it is recommended most strongly to provide mechanical shear connection in all beams designed for composite action.



(c) Tests demonstrated clearly the great toughness and reserve of strength and deflection after yielding possessed by these composite beams in which the shear connectors were adequate to provide almost full composite action to failure. In order to accomplish this it is necessary to provide shear connectors that will not yield or otherwise permit large slips until the ultimate capacity of beam is reached. The factor of safety of the shear connection should be greater than that of the beam itself.

(d) Shear connectors can be divided into three categories: (i) rigid connectors consisting of channels, angles or tees placed on end, (ii) flexible shear connectors consisting of plates or of channels with one flange welded to the beam, and (iii) bond connectors.

(e) Even though rigid connectors are superior to flexible ones from the point of slip, they have the disadvantage that they do not resist uplift as efficiently as the channel or Z type flexible shear connectors. Investigations revealed that flexible shear connectors behaved like dowels embedded in an elastic medium producing less bending moments in the web of the connector than in rigid ones and producing high bearing pressures behind and at lower portion of the connector. A rough method for design of channel shear connectors for ultimate loads has also been evolved, Appendix I.

(f) The ultimate moment carrying capacity of a composite steel and concrete beam with full interaction can be computed by assuming (a) compressive stress in concrete slab at failure is uniformly distributed and equal to cylinder strength, (b) concrete takes no tension, (c) full yielding occurs across the steel beam section, i.e., the stress reaches yield stress across the section [1].

### 4.3. Tests by G. A. Jones (1953)

4.3.1. Jones tested 12 encased beams with moderate cover 1 to 3 inches to study their behaviour under load. He observed that:

(a) When a steel joist acts together with the concrete encasing, its behaviour under load can be predicted accurately by the equivalent moment of inertia.

(b) When an encased beam is under load, the joist is in general subjected to a flexural and axial force, the flexural force being of much greater value than in normal reinforced concrete. The axial force depends on the percentage of concrete which is added and it is the rate of change of this force which gives rise to bond stress. The bond stresses are low compared to those met with in R. C. and are unlikely to govern the strength of the beam, and hence no mechanical aids to bond are required with small percentage of concrete. Allowable bond stresses may be taken as for R.C.

(c) At a maximum stress of 10 T per sq. in. in the tension fibres of the joist, the concrete cracks do not exceed 0.25 mm, thus giving adequate protection against corrosion. Under existing allowable stresses it is not possible to calculate by elastic methods a greater working load for the encased section, since concrete stresses are relatively too high.

## 5. PROPOSED PROCEDURE FOR DETERMINATION OF ULTIMATE STRENGTH OF ENCASED BEAMS

5.1. From the above investigations it is clear that encasement of steel beams not only adds to flexural rigidity of the steel joist, it also adds to its strength. In ordinary steel sections, the ultimate moment of resistance is not attained unless the compression flange is specially stiffened against buckling. Even nominal encasement will, however, stiffen the joist and help it to attain full plastic moment. Hence, in encased steel beams, the steel section can be assumed to attain its full plastic moment of resistance.

5.2. In addition to the stiffening action, encasement adds to the strength of the joist. The following assumptions, which are generally used in R. C. and steel designs, may be used to estimate this increased moment of resistance of encased beams.



### 5.3. Assumptions

(i) Concrete does not take any tension.

(ii) The ultimate compression developed in the compression zone of concrete is given by $C_c = \alpha C_w b n_1 d_1$ and that this force acts at a depth of $0.4 n_1 d_1$ (Baker)[6] Fig. 1. The recommended values of $\alpha$ depend on the cube strength $[C_w]$, it being

$\alpha$ = 0.5 for cube strengths over 8000 lbs per sq. inch and above.

= 0.55 for cube strength between 6000 and 8000 lb per sq. in.

= 0.6 for cube strengths 4000 lbs per sq. in. and below.

(iii) Concrete fails at a limiting strain. A safe value of 0.002 to 0.0035 which is well above the strain at yield of steel (0.001 to 0.0015 approx.)

(iv) Plane sections remain plane even after bending.

(v) The stress strain curve of steel in compression is the same as in tension.

(vi) Stresses due to creep, plastic flow, etc., are small and even though they may affect the stresses of composite beam at elastic stages they do not affect the ultimate strength.

(vii) Encasement does not reduce the plastic moment of resistance of the steel joist. At failure, the joist is strained to its ultimate moment of resistance. This value can be calculated on the assumption that full yielding occurs across the section.

### 5.4. Basis of the Procedure

5.4.1. The extra compressive strength in the concrete in compression can be considered as equivalent to adding an additional area on the compression side of the steel joist. As suggested by Batho, the joist may be considered as under the influence of a flexural and axial force, the latter being provided by the concrete in compression. The magnitude of the compressive force in concrete can be evaluated from any of the ultimate strength theories in concrete as that due to Prof. A. L. L. Baker.

5.4.2. The addition of this extra compression area will produce a shift in the neutral axis of the plain steel beam; the shift will depend on the magnitude of the concrete cover and can be evaluated from the condition that, in bending, the total tension should be equal to the total compression. Depending on the magnitude of the concrete cover, the neutral axis may shift to a position inside the joist or outside the joist. In the latter case, the joist acts like a reinforcement as in an R. C. beam and protection against bond failure may be difficult. In the more usual case, the neutral axis at failure will be inside the joist either in the web or flange. In such cases protection against bond failure can be easily provided for by mechanical shear or bond connectors. This case is recommended in all constructions.

5.4.3. Then the total ultimate moment of resistance of the encased joist is obtained by taking moments of the tensile and compressive forces about the new neutral axis.

5.4.4. Any assumption regarding stress distribution across the steel joist may be used (as, for example, the stress varying as the stress curve for steel) but the assumption of full plastic distribution of stress across the section is both simple and accurate for all practical purposes for rolled steel sections. With this assumption, *advantage may also be taken* of the tabulated value of the "plastic moment of resistance of the steel joists under compressive load" as that compiled by Horne[2]. Table 1 gives these values. Then the total ultimate moment of resistance is equal to the modified plastic moment of resistance due to shift of neutral axis plus the resisting moment due to the compressive force in concrete.

### 5.5. Procedure for Calculation (full Plastic Distribution of Stress in Joist):

**5.5.1. Step (1) Calculation of shift of neutral axis:** (Fig. 1)

Compression on concrete $= \alpha\, C_w\, b\, n_1\, d_1$

But $n_1\, d_1 = g - s$

where shift in neutral axis $= s$

Equating forces $\alpha\, C_w\, b\, n_1\, d_1 = (2\,fy) \times$ [area of steel in the shifted distance.]

$= 2\,fy\,(s.t_1)$ where $t_1$ = thickness of web. (Fig. 2).

From this the value of the depth of neutral axis can be evaluated.



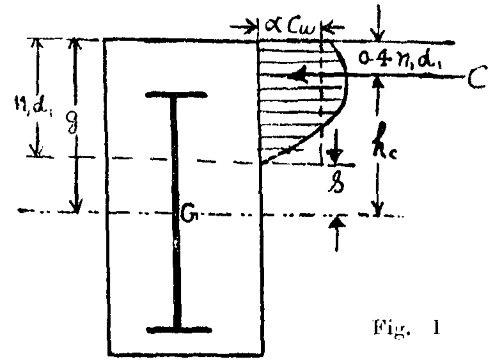


Fig. 1

**5.5.2. Step (2) Determination of modified plastic moment resistance of joist (modified due to axial force):**

Axial comp. force $= C_c = \alpha\, C_w\, b\, n_1\, d_1$

Ultimate axial force in joist $= fy.\,A$

$$n = \frac{C_c}{fy.\,A}. \quad \text{(in Table 1)}$$

Then the value of $M''_p$ (modified plastic moment of resistance due to axial force) is a function of $n$ and can be read off from Horne's tables. — Refer Table 1.

**5.5.3. Step (3) Calculation of total moment of resistance**

Total moment of resistance of encased joist $= M''_p + \triangle M$ (due to compressive force in concrete).

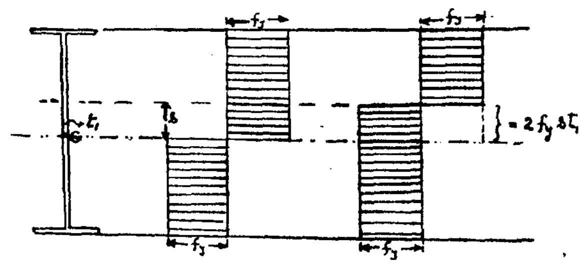


Fig. 2

$\triangle M$ = Moment of the compressive force about the C. G. of the steel section $= C_c.\,h_c$, Fig. 1.

TABLE I

PLASTIC MODULI OF B·S· SECTIONS - MAJOR AXIS
(after M.R. Horne)

| Beam Size | Area (in²) | Plastic Modulus (Major Axis) in³ | Reduced Values of P.M under axial load: Lower Value of n | Change at n= | Higher Value of n |
|---|---|---|---|---|---|
| 24"×7½" | 27.94 | 245.6 | $245.6-342.3n^2$ | 0.446 | $26.91(1-n)(11.46+n)$ |
| 22"×7" | 22.06 | 177.2 | $177.2-243.4n^2$ | 0.458 | $18.16(1-n)(12.36+n)$ |
| 20"×7½" | 26.19 | 194.3 | $194.3-285.7n^2$ | 0.409 | $23.63(1-n)(10.08+n)$ |
| 20"×6½" | 19.12 | 141.8 | $141.8-203.0n^2$ | 0.430 | $14.63(1-n)(12.07+n)$ |
| 18"×8" | 23.53 | 163.8 | $163.8-276.7n^2$ | 0.339 | $18.07(1-n)(10.72+n)$ |
| 18"×7" | 22.09 | 148.3 | $148.3-221.8n^2$ | 0.339 | $18.05(1-n)(10.01+n)$ |
| 18"×6" | 16.18 | 108.1 | $108.1-155.8n^2$ | 0.425 | $11.36(1-n)(11.82+n)$ |
| 16"×8" | 22.06 | 138.6 | $138.6-253.5n^2$ | 0.304 | $15.91(1-n)(10.09+n)$ |
| 16"×6"(H) | 18.21 | 106.23 | $106.23-150.6n^2$ | 0.429 | $14.25(1-n)(9.22+n)$ |
| 16"×6"(L) | 14.71 | 88.79 | $88.79-135.1n^2$ | 0.393 | $9.41(1-n)(11.49+n)$ |
| 15"×6" | 13.24 | 75.20 | $75.20-115.2n^2$ | 0.390 | $7.70(1-n)(11.88+n)$ |
| 15"×5" | 12.36 | 66.96 | $66.96-90.9n^2$ | 0.463 | $7.94(1-n)(10.67+n)$ |
| 14"×8" | 20.59 | 114.59 | $114.59-230.3n^2$ | 0.269 | $13.88(1-n)(9.38+n)$ |
| 14"×6"(H) | 16.78 | 88.43 | $88.43-140.7n^2$ | 0.363 | $12.08(1-n)(8.72+n)$ |
| 14"×6"(L) | 13.59 | 72.53 | $72.53-115.4n^2$ | 0.368 | $8.07(1-n)(10.74+n)$ |
| 13"×5" | 10.30 | 50.28 | $50.28-75.7n^2$ | 0.398 | $5.55(1-n)(11.05+n)$ |
| 12"×8" | 19.12 | 92.46 | $92.46-212.5n^2$ | 0.226 | $12.00(1-n)(8.56+n)$ |
| 12"×6"(H) | 15.89 | 72.72 | $72.72-126.2n^2$ | 0.319 | $10.83(1-n)(7.80+n)$ |
| 12"×6"(L) | 13.00 | 60.46 | $60.46-105.7n^2$ | 0.322 | $7.37(1-n)(9.58+n)$ |
| 12"×5" | 9.45 | 42.50 | $42.50-63.7n^2$ | 0.400 | $4.71(1-n)(11.02+n)$ |
| 10"×8" | 16.18 | 65.65 | $65.65-163.5n^2$ | 0.205 | $8.725(1-n)(9.27+n)$ |
| 10"×6" | 11.77 | 46.74 | $46.74-96.2n^2$ | 0.259 | $6.049(1-n)(8.73+n)$ |
| 10"×5" | 8.85 | 33.68 | $33.68-54.4n^2$ | 0.358 | $4.138(1-n)(9.70+n)$ |
| 10"×4½" | 7.35 | 28.07 | $28.07-45.1n^2$ | 0.363 | $3.174(1-n)(10.58+n)$ |
| 9"×7" | 14.71 | 53.18 | $53.18-135.2n^2$ | 0.197 | $8.092(1-n)(7.18+n)$ |
| 9"×4" | 6.18 | 20.85 | $20.85-31.8n^2$ | 0.389 | $2.517(1-n)(10.04+n)$ |
| 8"×6" | 10.30 | 32.94 | $32.94-75.7n^2$ | 0.224 | $4.670(1-n)(7.82+n)$ |
| 8"×5" | 8.28 | 25.80 | $25.80-48.9n^2$ | 0.286 | $3.604(1-n)(8.19+n)$ |
| 8"×4" | 5.30 | 16.02 | $16.02-25.0n^2$ | 0.376 | $1.882(1-n)(10.26+n)$ |
| 7"×4" | 4.75 | 12.90 | $12.90-22.5n^2$ | 0.323 | $1.519(1-n)(9.93+n)$ |
| 6"×5" | 7.37 | 17.00 | $17.00-33.1n^2$ | 0.271 | $2.873(1-n)(6.67+n)$ |
| 6"×4½" | 5.89 | 13.46 | $13.46-23.4n^2$ | 0.325 | $2.072(1-n)(7.53+n)$ |
| 6"×3" | 3.53 | 8.084 | $8.084-13.56n^2$ | 0.337 | $1.094(1-n)(8.69+n)$ |
| 5"×4½" | 5.88 | 11.653 | $11.653-29.81n^2$ | 0.192 | $2.027(1-n)(6.25+n)$ |
| 5"×3" | 3.26 | 6.318 | $6.318-12.03n^2$ | 0.283 | $0.929(1-n)(7.76+n)$ |
| 4¾"×1¾" | 1.91 | 3.335 | $3.335-5.06n^2$ | 0.382 | $0.538(1-n)(7.43+n)$ |
| 4"×3" | 2.94 | 4.539 | $4.539-8.00n^2$ | 0.264 | $0.7634(1-n)(6.70+n)$ |
| 4"×1¾" | 1.47 | 2.153 | $2.153-3.17n^2$ | 0.401 | $0.3266(1-n)(8.00+n)$ |
| 3"×3" | 2.52 | 2.979 | $2.979-7.95n^2$ | 0.179 | $0.5662(1-n)(5.68+n)$ |
| 3"×1½" | 1.18 | 1.309 | $1.309-2.16n^2$ | 0.334 | $0.2411(1-n)(6.82+n)$ |

NOTE
(L) = Light; (H) = Heavy; $n = \frac{N}{N_P}$ where N = Axial Force, $N_p$ = Full Plastic Thrust

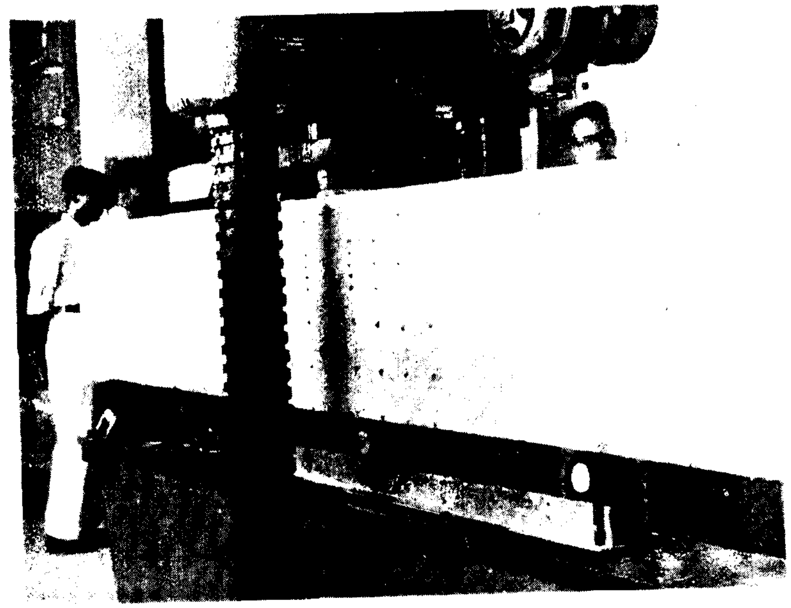

Photo 1

**12 in. × 6 in. encased joist with 5 in. top cover beam No. 5, under test**

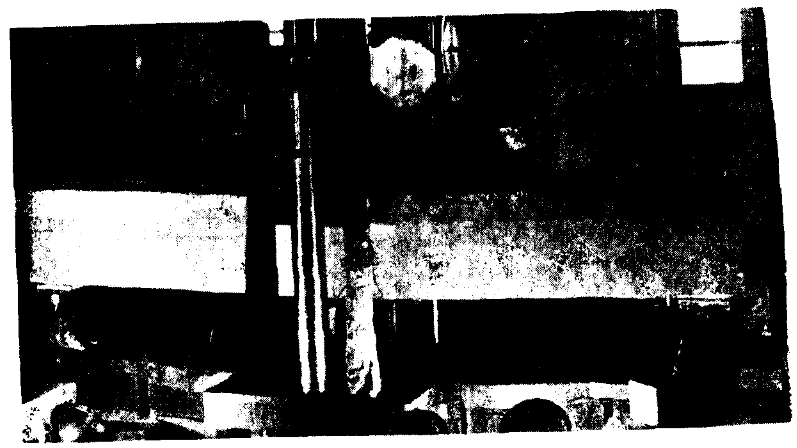

Photo 2

**12 in. × 6 in. encased joist with 5 in. × 36 in. slab on top (Tee beam) beam No. 6, under test**

Thus $M'_{pe} = M''_p + C_c\ h_c$

Where $M'_{pe}$ = Plastic moment of resistance of encase joist.

$M''_p$ = Modified plastic moment of resistance du to axial compression.

5.5.4. If conditions are such that the neutral axis falls out side the joist, the same principles may be used for computatioı of the ultimate moment of resistance. In this case, the ultimat moment of resistance will be, as in R. C. design, the product o the total compressive force in concrete and its distance fron the centre of gravity of the steel sections. As remarked earlier, a the maximum shear stresses in a beam occur at the neutral axi and as the neutral axis in this case falls above the joist in the concrete, prevention of bond and shear failure in this case will be difficult.

## 6. ILLUSTRATIVE EXAMPLE

6.1. The following example illustrates the suggested procedure of computation.

6.2. A rectangular encased joist is made out of a B.S. 12 in. × 6 in. section @ 44 lb with 5 in. cover on compression side and 2 inches on all other sides. Determine its ultimate moment of resistance if yield strength of steel is 19.0 tons per sq. inch and cube strength of concrete is 3650 lb per sq. in. Thickness of web = 0.4 inches. Area of steel section = 13 sq. in., Fig 3.

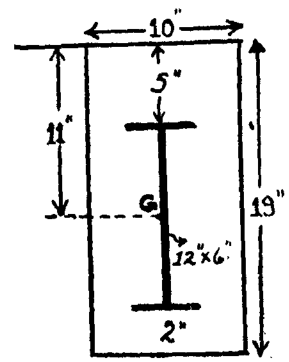


Fig. 3

(1) Location of new neutral axis at ultimate load. Lets be the shift of the neutral axis,

Then $\alpha\ C_w\ b\ n_1 d_1 = 2\ fy\ si$,

$$0.6 \times \frac{3650}{2240} \times 10 \times (n_1 d_1) = 2 \times 19.0 \times 0.4\ [11 - n_1 d_1]$$

$$n_1 d_1 = \frac{15.2 \times 11}{24.96} = 6.7 \text{ inches}$$

(2) Total compression in concrete at ultimate load } $9.76 \times 6.7 = 65.4$ tons

(3) Ultimate moment of resistance of joist under axial load of 65.4 tons from Horne's tables

$$= \frac{N}{N_p} = \frac{64.5}{13 \times 19.0} = 0.261$$

$Z''_p = 60.46 - 105.7\ n^2$ [from Horne's tables — Table 1]

$= 53.26$

$M''_p = 53.26 \times 19.0 = 1012$ in. tons

$\triangle M$ due to concrete = Moment about C.G. of steel

$= C_c\ h_c$

$h_c = 4.3 + (0.6 \times 6.7)$

$= 8.3$ inches.

$\triangle M = 65.4 \times 8.3 = 543$ in. tons

$M'_{pc} = M''_p + \triangle N$

$= 1555$ in. tons

$M_p$ of steel joist $= 60.46 \times 19.0 = 1148$ in. tons

(4) Per cent increase $= \frac{407}{1148} \times 100 = 35.4$ per cent.

In an actual test, the beam tested under the above condition failed at a moment of 1700 in. tons. The increase over the calculated value is due to part of the steel beam reaching the increased stress due to strain hardening. The test fully confirms the validity of the calculations. Photo 1 shows the particular beam under test.



## 7 RESULTS OF EXPERIMENTAL VERIFICATION

The suitability of this method of calculation was experimentally verified by a number of actual tests in the Civil Engineering Llaboratory, Indian Institute of Technology, Kharagpur. Table 2 gives summary of some of the tests so far completed. Photo 2 shows beam No. 6 under test.

TABLE 2

**Comparison of test results with computed values**

| Beam No. | Description | Ultimate moment | |
|---|---|---|---|
| | | Calculated by proposed procedure | Value determined by actual test |
| (1) | (2) | (3) | (4) |
| 1. | 4 in. × 1¾ in. joist with 1 in. cover on all sides | 50 in. ton | 63 in. ton |
| 2. | 4 in. × 1¾ in. joist with 2½ in. cover on top and bottom and 1 in. cover on sides | 65 ,, | 80 ,, |
| 3. | 5 in. × 3 in. joist with 3 in. cover on top and 2 in. cover on sides and bottom | 192 ,, | 200 ,, |
| 4. | 6 in. × 3 in. joist with 2 in. cover on all sides | 194 ,, | 190 ,, |
| 5. | 12 in. × 6 in. joist with 5 in. cover on top and 2 in. on sides | 1555 ,, | 1700 ,, |
| 6. | 12 in. × 6 in. joist with a slab 5 in. × 36 in. on top and 2 in. cover on sides | 2141 ,, | 2460 ,, |
| 7. | 6 in. × 3 in. joist unencased 10 ft span and third point loading | 142 ,, | 104 in. tons failed in buckling in compression flange |

## 8. CONCLUSIONS

8.1. The above discussions show that encasement considerably increases the strength and stability of steel beams at elastic and ultimate stages. Table 3 shows the increase in ultimate strength of standard beams encased with minimum cover for fire precaution. The increase in strength for properly designed beams with large cover and for tee beams will be much higher. For indeterminate structures as in continuous beams this increase will be very pronounced.

TABLE 3

**Showing increase in ultimate moment ($M_p$) of resistance of steel joists encased with 2 inch cover**

Assumed yield stress of steel = 15.25 tons per sq. in.
Cube strength of concrete = 2250 pounds per sq. in.

| Joist | $M_p$ of joist inch tons | $M_p$ of encased sections inch tons | Approx per cent increase |
|---|---|---|---|
| (1) | (2) | (3) | (4) |
| 24×7½ | 3740 | 4315 | 16 |
| 22×7 | 2700 | 3184 | 18 |
| 20×7½ | 2960 | 3380 | 14.2 |
| 20×6½ | 2165 | 2525 | 16.6 |
| 18×8 | 2500 | 2840 | 13.6 |
| 18×7 | 2260 | 2585 | 14.4 |
| 18×6 | 1650 | 1926 | 16.8 |
| 16×8 | 2100 | 2370 | 12.9 |
| 16×6 (Heavy) | 2620 | 1868 | 15.3 |
| 16×6 (Light) | 1350 | 1593 | 18.0 |
| 15×6 | 1150 | 1350 | 17.8 |
| 15×5 | 1020 | 1215 | 19.1 |
| 14×8 | 1740 | 2030 | 16.6 |
| 14×6 (Heavy) | 1350 | 1543 | 12.5 |
| 14×6 (Light) | 1110 | 1288 | 16.0 |
| 13×5 | 766 | 930 | 21.4 |
| 12×8 | 1410 | 1583 | 12.3 |
| 12×6 (Heavy) | 1110 | 1264 | 14.0 |
| 12×6 (Light) | 920 | 1071 | 16.4 |
| 12×5 | 648 | 780 | 20 |
| 10×8 | 1000 | 1126 | 12.6 |
| 10×6 | 715 | 823 | 15.1 |
| 10×5 | 514 | 615 | 19.7 |
| 10×4½ | 430 | 510 | 18.6 |
| 9×7 | 812 | 908 | 11.8 |



TABLE 3 (*Contd.*)

| Joist | $M_p$ of joist inch tons | $M_p$ of encased sections inch tons | Approx. per cent increase |
|---|---|---|---|
| (1) | (2) | (3) | (4) |
| 9×4 | 320 | 396 | 23.8 |
| 8×6 | 502 | 579 | 15.3 |
| 8×5 | 393 | 464 | 18.0 |
| 8×4 | 244 | 308 | 26.2 |
| 7×4 | 197 | 231 | 17.8 |
| 6×5 | 260 | 316.8 | 22.0 |
| 6×4½ | 206 | 261.4 | 26.8 |
| 6×3 | 123 | 160.5 | 30.2 |
| 5×4½ | 178 | 216.5 | 21.6 |
| 5×3 | 96.5 | 126.5 | 31.0 |
| 4¾×1¾ | 51.0 | 73.0 | 43.0 |
| 4×3 | 69.5 | 94.0 | 35.0 |
| 4×1¾ | 33.0 | 56.0 | 70.0 |
| 3×3 | 45.5 | 62.4 | 37.0 |
| 3×1½ | 20.0 | 34.0 | 70.0 |

8.2. The procedure proposed is simple and quick and can also be used for speedy calculation of ultimate strength of encased steel beams for design of shear connectors in highway bridge floors.

ACKNOWLEDGMENTS

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## Appendix IX

### A Note on Design of Channel (flexible) Shear Connectors for Composite I Beams.

The following method after the method proposed by Viest and Seiss[1] may be used for design of shear connectors.

**Assumptions**

(a) Shear connectors are required not only to transfer horizontal shear from concrete to steel, but also to tie the slab and the beam together in the vertical direction. Channel shear connectors fulfil these functions.

(b) These shear connectors are rigidly welded to the top of the encased joists. When the loads are applied to the beam the greatest pressure is exerted against the concrete at the base of the connector. The flexible web deflects as a dowel. Thus the distribution of pressure is as shown in Fig. 4.

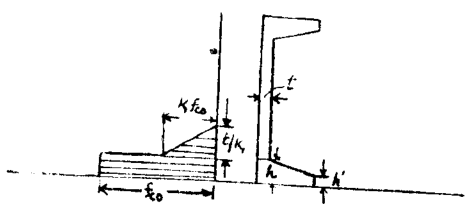


Fig. 4

(c) Failure of shear connector occurs either by crushing of concrete behind it or by the material of the shear connector reaching yield stress.

Theoretical studies have shown that the maximum stress in concrete behind the shear connector is given by

$$Fco = \frac{Q}{(h+\frac{1}{2}t)w} \quad \ldots \quad (1)$$

$Q$ = Total load on a shear connector

$h$ = Maximum thickness of channel flange

$t$ = Thickness of the web of the channel

$w$ = Length of channel shear connector across the beam.



Similarly, the maximum stress in the channel shear connector is given by

$$F\max = \frac{Q}{k_1\,(h+\frac{1}{2}t)w} \quad \ldots \quad (2)$$

When $K_1 = \dfrac{1.6}{\dfrac{fco}{f'c} + 1.8}$

$f'c$ = Cylinder (6 in. × 12 in.) strength of concrete.

As failure of shear connector occurs when $f$ max reaches yield stress ($f\,yp$), we can write *equation*.

$$\frac{Q\,yield}{(h+\frac{1}{2}t)wf'c} = -0.9 + \sqrt{0.81 + 1.6\frac{fyp}{f'c}} \quad \ldots \quad 2\,(a)$$

which can be approximated for the usual values of $fyp$ = 30,000 and $f'c$ = 2500 to 5000 pounds per sq. in. to

$$\frac{Qyp}{(h+\frac{1}{2}t)\,wf'c} = \sqrt{\frac{fyp}{f'c}} \quad \ldots \quad 2\,(b)$$

without much error.

If we assume that all the force is transferred through the shear connectors, then in the elastic stage

$$Q = \frac{V\,A_c\,y_c}{nI} \cdot s. \quad \ldots \quad (3)$$

where

$s$ = Spacing of shear connectors

$V$ = Vertical shear on a composite beam

$A_c$ = Cross sectional area of the portion of the concrete above the steel joist.

$y_c$ = Distance between the centre of gravity of the concrete slab and neutral axis of the composite section

$I$ = Moment of inertia of composite T beam computed from the area of steel beam and transformed area of the slab

$n = \dfrac{E_s}{E_c}$

We further assume that the value of $Q$ increases in direct proportion to the shear on the section. This will give approximately correct values on shear connectors located at portions where yielding has not occurred and somewhat lower values where the yielding has occurred ; but yielding generally occurs at a few places and the overstressing of a few connectors is not considered critical.

$$\text{Thus} \left[\frac{Q \text{ yield}}{Q \text{ design}}\right] \text{ of connector} = \left[\frac{V \text{ ultimate}}{V \text{ design}}\right] \text{ of beam}$$

where $V$ = Shear force ... (4)

It is specified that for composite action the shear connectors should be safe to ultimate failure of the beam so that equation 4 can be written as

$$\left[\frac{Q \text{ yield}}{Q \text{ design}}\right] = \left[\frac{M \text{ (ultimate)}}{M \text{ (design)}}\right] \quad \ldots \quad 4\text{ (a)}$$

where $M$ = Bending moment.

The value of $M$ (ultimate) can be calculated by the proposed procedure. With the help of equation (4), (2) and (1), the maximum force in the shear connector ($Q_{yp}$), the dismensions of the shear connector ($w$, $h$, $t$) and the maximum stress ($fco$) in the concrete can be checked.

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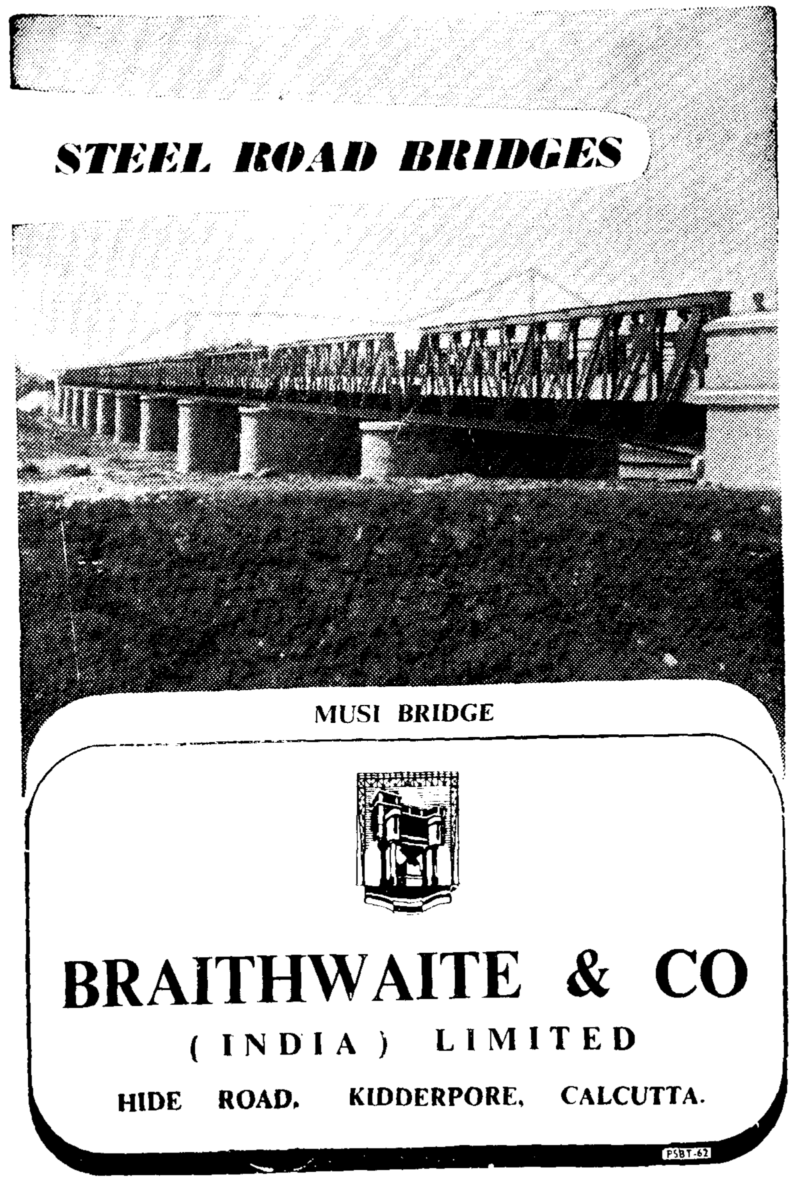
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