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NOTICES AND ANNOUNCEMENTS

TRAFFIC STUDY WEEK - JULY 1959

A Traffic Study Week is being organised at Bombay in July 1959 under the auspices of the ECAFE. The exact dates and programme will be announced shortly. In the meantime, members are requested to contribute Papers on the following subjects for discussion at the proposed Traffic Study Week :

I. **Accident Studies**:

(a) Traffic accident statistics: accident report forms, reporting procedure and filling of reports;

(b) Analysis of causes of accidents: study of traffic accidents according to type of collision, type of vehicle, road condition, driver's age, traffic violations, and other factors;

(c) Elimination of 'black spots'; experiments with various methods, and 'before-and-after' studies assessing results.

II. **Traffic Operations:**

(a) Regulation of movement: systems of speed limits and speed zoning in rural and urban areas;

(b) Studies and field measurements of intersection and road capacities under mixed traffic conditions in rural and urban areas;

(c) Geometrics of roads and their effect on traffic flow and accidents.

III. Road user behaviour and methods for improving the same.

IV. Future trends and traffic prediction for planning urban and rural roads.

2. **Procedure**: Members who intend to contribute Papers are requested to intimate the fact direct to Shri D. D. Suri, I.A.S., Deputy Secretary, Govt. of India, Ministry of Transport and Communications, Department of Transport, Transport Wing,

North Block, New Delhi, and to send a short synopsis of the proposed Paper to him as early as possible.

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

4. The complete Paper should be submitted *before the end of February 1959.*

Paper No. 210

"BRICK PAVEMENT IN HIGHWAY CONSTRUCTION"

By

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&

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CONTENTS

SYNOPSIS

The Paper outlines the possibilities of using bricks in highway construction in India. Brick roads are already existing in Western countries like Holland and Germany.

A Bull's kiln was established on Lucknow-Sultanpur road to produce bricks of superior quality to be used on this road. The factors affecting the quality of bricks are discussed and the effect of increasing the duration of firing on the quality of bricks is shown. The temperatures obtained in the kiln and the results of tests on bricks and brick ballast are discussed.

Specifications adopted in laying different types of brick pavements and their behaviour are described. The behaviour of surface dressing and premix carpet on brick pavement is also discussed and broad conclusions given.

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1. INTRODUCTION

1.1. The highways at present are carrying heavier and more intense traffic than ever before due to development of road transport. The maintenance of water-bound kankar roads is thus becoming uneconomical due to the necessity of frequent renewals of the top coat and also due to increased repairs of patches, pot holes, ruts, etc., to maintain the riding surface in a satisfactory condition. Modernisation of such roads is thus becoming essential.

Modernisation usually consists of providing a top coat of stone metal 3 in. (loose) thick over an existing kankar surface and consolidating it with roller and then providing either one coat or two coats of surface dressing with the usual specifications. The one coat work consists of application of mexphalt and stone grit and the two coat work consists of application of tar and grit followed by mexphalt and grit.

1.2. These specifications are quite costly as the average cost of modernisation comes to Rs 29,000 for one coat work per mile. In U.P. the stone metal is available from quarries in the North (Himalayas) or in the south (Vindhyan range). It thus becomes costly to transport stone metal from long distances and at the same time delays occur due to limited transport facilities available on railways.

1.3. Due to the necessity of using a cheaper and a suitable road material in modernisation work and to provide more and more miles of such roads, overburnt brick ballast produced from bricks has been used quite extensively in place of stone metal in Uttar Pradesh for the last several years. Hundreds of miles of such roads have been laid and these have been behaving quite satisfactorily under different intensities of traffic[1, 2]. The specifications adopted which provide for bituminous surfacing soon after consolidating overburnt brick ballast are no more in an experimental stage.

1.4. In order to reduce the cost of modernisation still further, more and more experiments and trials are being carried out.

1.5. Apart from providing dustless and durable surfaces as envisaged in the modernisation programme, it is becoming more essential to provide all-weather cheaper roads to connect village to village as well as to connect villages to National and State Highways and Major District Roads in order to develop their economy. Construction of stabilised soil roads is coming into vogue. There are, however, some difficulties hindering its development. The main one is the necessity of having a field laboratory with experienced staff for testing the soil samples. Further, the technique is still not well known to many field engineers and they are hesitant to use it. More supervision is necessary in this work.

1.6. The other alternative which appears feasible and economical is the utilisation of high compressive strength bricks for laying pavement direct on a soil subgrade with and without sand cushion. It is also possible to use such bricks for modernisation purposes by laying them on an existing kankar surface with sand or earth cushion and then leaving the surface as such under traffic or providing a black top surface over brick pavement with surface dressing or premix carpet.

1.7. Bricks have been one of the principal paving materials for road construction for the last hundred years. Actually their use for road paving goes back into remote ages and there is evidence throughout this country of buildings, palaces and temples paved with durable bricks. Vitrified brick roads built in Holland during Napoleon's time are in service today. The service given by brick roads 30 or more years old in practically every city of the United States has been really remarkable.[3]

1.8. Some of these brick pavements were laid on natural soil without foundations. In Holland, due to bad subsoil conditions, the road pavement becomes very uneven due to large settlements in the first five or six years of laying a new fill. Brick roads being cheapest to repair, Holland still prefers to keep pavements in bricks on all new roads, even though the cost of brick roads is almost the same as that of a cement concrete or asphalt pavement. Holland has thus laid considerable lengths of brick pavement both on primary and secondary roads.[4]

1.9. In India, between the year 1880 and 1920, vitrified bricks were manufactured at the Govt. brick field at Chetpat, Madras, and they were giving excellent service in the floors of stables of veterinary dispensaries and Police quarters even in the year 1936.[3]

1.10. Experimental brick trackways were laid on Delhi-Mathura Road during the year 1941 in order to see if such a pavement could be used to take occasional heavy motor traffic as an emergent measure in places where usual road materials may not be available and water is scarce.[5] Brick-on-edge was laid on compacted earth subgrade (after the old metalling and soling were removed) in different specifications. Observation of the behaviour of these surfaces indicated that brick trackways gave satisfactory performance with and without black-top surface on top, after the joints between the bricks had been compacted with sand or earth under traffic.

1.11. Laboratory investigations carried out on some soils have indicated that bricks possessing high compressive strength of the order of 10,000 lb per sq. in. can be produced provided temperature and period of burning are controlled.[6] Bricks produced at high temperature and burnt for longer duration possess high strength as compared to bricks produced by normal burning.

1.12. The possibilities of producing high compressive strength bricks and using them in road pavement are thus quite great, provided such bricks can be made available economically at the road side.

1.13. Investigations into the possibilities of producing such bricks in the usual type of Bull's kiln have been carried out by the U.P. P.W.D. Research Institute. Bricks were burnt for longer duration to achieve better results. These bricks have been used in laying brick pavement on Lucknow-Sultanpur Road in 1 mile 4½ fgs. length as an experimental measure. The results of investigations carried out on the production of bricks in the kiln and on the behaviour of different types of brick pavements laid with and without black top surfaces are reported and discussed in the following pages.

2. ADVANTAGES AND DISADVANTAGES OF BRICK ROADS

2.1. There can be only one disadvantage in brick roads as compared to cement concrete or asphalt roads, viz., they are not so even to ride upon. But unevenness in brick roads per unit area is large in number but small in magnitude.[8] Further, the condition of kankar roads which is the usual form of road construction adopted in this State in the beginning, can be worse if proper maintenance and renewals are not done in time.

2.2. The advantages of brick roads are as follows :

(1) The surface is dust free.

(2) While constructing new roads it is not necessary to construct unnecessarily high embankments as any portion that gets submerged during subsequent rains can be raised at the time of relaying of bricks without adding any extra cost to the work and without damaging the work (physically or financially) previously done.

(3) Relaying can be carried out at any time of the year and it can be so programmed that the village labour is provided with work during periods of slack season.

During relaying, the worn out surface of bricks can be put down or at sides which gives a new appearance to the pavement.

(4) Relaying need not be done in the rainy season, thereby saving the traffic the trouble of moving over muddy shoulders or patries as it happens at present when kankar miles are under renewal or stone metal or brick metal is being consolidated.

(5) Maintenance cost of a brick pavement is expected to be much less than that of kankar or black top surface.

The maintenance of brick pavement involves the replacement of crushed bricks occasionally and relaying of the brick pavement after an interval of some years. This cost is going to be quite low as compared to the cost of maintenance and renewal of kankar roads.

(6) In Holland, the kilns are established near river banks, and they obtain their soil for burning bricks from river beds. This has two advantages :

(a) Borrow pits made in the river get silted up during floods, thereby leaving no unsightly and unhygienic pits.

(b) Soil remains available every year from the same source and as such the kilns need not be abandoned or shifted. The capital initially invested on the kiln can thus be made to work for a longer period.

The only disadvantage in this procedure is the extra lead for burnt bricks. But as the kiln can serve a fairly wide area, this may not be a big disadvantage.

(7) It has been found that the California Bearing Ratio Value (C.B.R.)[8] of the subgrade under a brick road increases considerably more than under an asphalt or cement concrete road.

If, at a later stage, bricks are to be replaced for providing a better surface like that of asphalt or cement concrete a more firm subgrade is obtained.

3. PRODUCTION OF HIGH QUALITY BRICKS

3.1. Manufacture of bricks essentially consists of the following operations :

(i) Selection of suitable brick clay and site.

(ii) Winowing and preparation of clay.

(iii) Moulding and drying of bricks.

(iv) Firing and cooling of bricks.

3.2. Selection of Suitable Brick Clay

3.2.1. Not all soils yield good bricks. Suitable soil must, therefore, be selected for making good quality bricks and that is where the importance of testing the soil and finding out its properties lies. This saves labour and expenses involved as the tests reveal a good deal about the quality of bricks expected to be produced from a particular soil.[7] The tests can reveal if it is necessary to blend the soils from different strata or places in order to get the right type of raw materials.

3.2.2. Study of Soils

In considering the properties and tests required to be performed for suitability of brick clay, one is usually concerned with those tests which are associated with the workability of clays and firing behaviour. The drying and firing behaviour of brick clays is closely associated with the type and size of clay aggregate and associated constituents. The quality of fired products is dependent upon the maximum firing temperature, rate and duration of firing and proper firing atmosphere.

At present, we depend upon physical analysis of soils for selecting a suitable brick clay. The study of physical properties of soils consists in determination of particle size analysis, Atterberg limits, and shrinkage characteristics.

Based on the results of investigations carried out in the U.P., P.W.D. Research Institute correlating the physical properties of soils with the quality of fired bricks. specifications for soils for producing good quality bricks have been drawn.[8] By means of these tests, it is possible to reduce the uncertainties inherent in soils.

It is considered that chemical and mineralogical analysis of soils should also be carried out as properties and firing behaviour can be determined to a great extent by the nature and amount of clay minerals present.

3.2.3. Firing Behaviour

At high temperatures, silicate formation occurs and the clay particles fuse, A certain amount of vitrification and glass formation also occurs. These changes give new properties to the bricks. They become hard and compact. The "fusible glass" formation occurs due to the inter-action between the finer particles of clay and sand grains which binds together the coarser particles of clay constituents. This is the chief source of strength and density of bricks.

In burning of common building bricks, attempt is made to form only a small amount of this fusible glass. If clay products are burnt to a still higher temperature, a larger quantity of this new glassy materials is formed and the products are said to be "vitrified". Beyond a certain limit, vitrification results in the general softening of the clay and the products loose their shape.[9] Controlled vitrification, however, imparts strength and density. and the products are almost non-porous. While producing bricks for use in road construction, the aim must be to produce vitrified bricks.

The firing behaviour of a particular soil is estimated in the laboratory by preparing clay after powdering and intimately mixing it with water. The quantity of water mixed is the quantity actually used in the field. After investigation, it was found that the percentage of moisture used is near the liquid limit or 10 per cent less than the liquid limit. Small size bricks are then moulded, dried and then fired in electric muffle furnace at a desired temperature and for required duration. Bricks are fired at temperatures between 900° - 1000° for 5 to 7 hours. The bricks so produced are tested for compressive strength and water absorption as per B.S. specification [10] or I.S. specification [11].

The values obtained indicate the suitability or otherwise of the soil. At present, the U.P., P.W.D. Research Institute is classifying bricks according to tentative specifications which stipulate compressive strength of 3000 lb per sq. in. and above and water absorption of less than 1/6th the weight for first-class bricks[8]. These requirements are lower than those specified in the I.S.I. specification.[11]

3.2.4. Suitability of Soil for Kiln on Lucknow-Sultanpur Road

In the present case, a preliminary survey was carried out of the possible locations of the brick kiln sites on Lucknow-Sultanpur Road. Soils obtained from sites located in different miles were obtained and tested in the laboratory as per details outlined above.[6] The results indicated that the soil from furlong 4 of mile 22 was suitable for producing good quality bricks. The bricks burnt at 1000°C for 7 hours gave a compressive strength of 10,000 lb per sq. in.

Trial pits dug out at this place, however, indicated that requisite soil was not available in sufficient quantity.

Further investigations were, therefore, carried out and soil samples from mile 22, furlong 5 and mile 23 were collected and tested. Results given in Tables 1 and 2 revealed that the soil from mile 22, furlong 5 gave quite satisfactory results. The bricks fired at 950°C for 7 hours gave acompressive strength of 10,500 lb per sq. in.[12]. Trial pits dug out at several places on this site revealed the presence of the desired type of soil in layers of 3 to 4 ft.

The site was also higher than the country level, and the water table was low. It was, therefore, decided to set up a kiln of .6 lakh capacity per round on this site. Brief details of the kiln are given in Appendix A.

3.3. Winowing and Preparation of Clay

3.3.1. In ancient practice, clay used to be exposed to weather for some time before the actual brick-making was started. This practice is not very common now in this part of the country. Weathering for sufficiently long periods breaks down the clay in small lumps and increases the plasticity.

3.3.2. In the present case, the clay was dug out, watered and allowed to stand for 6 hours. It was then tempered by kneading under feet. The clay puddle was again kneaded next day. After kneading, the clay puddle was churned and the process continued till the clay became a homogeneous mass with a uniform consistency and possessing the required amount of plasticity for moulding.

3.3.3. Pug Mill

Pug mills are used for preparing clay of more uniform consistency and plasticity. The clay after kneading is fed into the mill from the top and after churning it comes out at the bottom. As an experimental measure, some bricks were got moulded by a vertical pug mill driven by bullocks.

3.4. Moulding

3.4.1. Out of the three principal methods of forming bricks, viz., soft mud process by hand, stiff mud process and semi-dry (pressed bricks) process, the soft mud process was adopted, being simpler and economical. The other two processes would each have required some machinery but would have given uniform and better bricks.

3.4.2. In the soft mud process both table moulding and site moulding were adopted, using sand for sprinkling the inside of moulds. As the bricks were to be used in the road pavement, no frog was provided.

3.5. Firing

3.5.1. Selection of Kiln

As stated before, Bull's kiln was selected for producing bricks as it is the normal type of kiln used in the State and the mistries and workers are well acquainted with its working.

Hoffmann type kiln is costly to set up but is suitable for burning bricks throughout the year. In the present case, only a limited quantity of bricks was required to be burnt and hence Bull's kiln appeared to be quite economical and suitable. Further, it was thought that the results of investigations could be used on a large scale on this type of kiln, if successful.

3.5.2. The bricks after drying were loaded in the kiln. Sixteen layers of brick-on-edge were stacked, over which two layers of flat bricks were kept as covering. The top was also covered by 3 in.–4 in. of earth or rubbish to prevent loss of heat. Eight combustion chambers were provided in each compartment. The arrangement of loading of bricks and the procedure of firing was the same as adopted in a Bull's kiln.

3.5.3. The firing of bricks in the kiln is normally done for 18 hours, the rate of feed of coal being at 15 minutes interval.

3.5.4. Attempts were made in the beginning to increase the temperature inside the kiln by reducing the interval of coal firing to 12 minutes, keeping the rate of feed of coal the same. It was noted that the whole of the coal was not burning and the temperature was lowered. The quantity of coal was then reduced proportionately but this also did not help and hence this procedure was not adopted and the usual firing at 15 minutes interval was continued.

3.5.5. As an experimental measure, attempts were made to produce high compressive strength bricks touching the vitrified range, by burning the bricks for extra period than the normal firing interval. Thus the bricks were burnt for ½, 1, 1½, 3, 6, 9, 12, 24, 36, 48 hours extra period. In the last compartment before close of firing, some bricks were even fired and burnt for 7 days continuously.

3.5.6. It was observed during this extra burning that, sometimes, the coal being fed into the holes was not burning fully.

thereby tending to reduce the temperature. The rate of feed of coal had thus to be reduced and thus lesser quantity of coal was consumed during prolonged firing than expected.

The holes had to be cleared of unburnt coal occasionally by means of iron bars.

3.5.7. After unloading the bricks it was noted that bricks from the top 2 layers were generally underburnt. As all bricks were required for pavement purposes, it was decided to reburn such bricks. Accordingly, these bricks were reloaded in the kiln and reburnt as usual. The quality of such bricks improved.

3.6. Temperature and Settlement Measurements

3.6.1. Temperatures inside the kiln were measured by means of immersion type pyrometer in the beginning. The temperature measured was at an approximate depth of 3 ft which was the length of the pyrometer. Later on, optical pyrometer was used for recording temperatures. This instrument proved very handy and useful. The operator used to measure the maximum temperature through the feeding holes which was dependent upon the brightness of the flame.

3.6.2. Study of the temperatures recorded brought out the following points:[13]

(i) Temperature under normal firing varied between 1030°C–1070°C.

(ii) Maximum temperature of 1120°C. was recorded in some holes.

(iii) Lower temperatures were recorded in the side holes than those in the central holes. Maximum temperatures in the central holes ranged between 1070°–1100°C. and in the side holes 1040°–1080°C.

(iv) When firing was done for longer period, the temperature inside the holes tended to lower down and increase during firing.

For example, while firing for 72 hours extra period, temperature when firing was started was 950°C. and temperature remained below 1000°C. after 28 hours of firing upto the close of firing. Maximum temperature reached in this hole was 1080°C., Plate I (A).

(v) In another hole, temperature when firing was started was 1020°C.; it rose to 1070°C, and then came down to 1000°, after some time. Temperature when firing was closed was 1040°C., Plate I (B).

3.6.3. In Holland, bricks possessing compressive strength of 8400 to 12600 lb per sq. in. are produced by using soil of a particular mechanical composition and by firing bricks at a higher controlled temperature and for longer duration. A typical temperature chart obtained during such firing is indicated in Table 3.

3.6.4. Such a control of temperature is not possible in an ordinary Bull's kiln unless some improvements in the design of the kiln and mode of stacking and firing of bricks are made and tried.

3.6.5. The temperatures attained inside the Bull's kiln are fairly high and it is expected that good quality bricks can be obtained if firing is done for longer duration under proper control.

3.6.6. Settlement

Settlement caused due to burning of bricks in the kiln was observed from time to time. It was noted that settlement under normal firing was about 9 in. in the second round and for extra firing a settlement upto $11\frac{1}{2}$ in. was observed.

In the first round the settlement recorded was slightly more, probably due to settlement of the floor during firing which did not happen in the second and subsequent rounds.

3.7. Consumption of Coal

3.7.1. Consumption of coal for normal burning and for extra burning is shown in Table 4. The normal consumption of coal was 25 tons per lakh of bricks during the first round of firing and 16 tons during the second round.

3.8. Cost of Bricks

3.8.1. About 9.23 lakh of brick were burnt in the kiln. The percentage of bricks of different categories was:

First class bricks	..................38.7 per cent
Straight overburnt bricks	...............17.0 per cent
Mis-shapened overburnt bricks	.........38.7 per cent
Second class and underburnt bricks	...5.6 per cent

3.8.2. The normal yield of bricks in a Bull's kiln is I class and overburnt 50 per cent; II class 30 per cent; III class 10 per cent and bats 10 per cent.

3.8.3. It will be seen that the percentage of mis-shapened overburnt bricks in this kiln was high as compared to that in an ordinary kiln. This was due to extra firing resorted to and also due to the fact that temperatures inside the kiln were not uniform throughout the depth, although this could not be confirmed by actual observation of temperatures at different depths. The percentage of second class and underburnt bricks was reduced considerably.

3.8.4. The cost of first class and straight overburnt bricks came to Rs 30 per thousand and of mis-shapen straight overburnt bricks to Rs 28 per thousand at the brick kiln site.

4. TESTS ON BRICKS AND BRICK METAL

4.1. Test on Bricks

4.1.1. The bricks produced from different chambers of the kiln burnt under normal conditions and for extra periods were tested in the laboratory for compressive strength and water absorption according to B.S. specification [10] and Indian Standards Specification [11].

4.1.2. The authorities in many countries using bricks for road pavement specify that bricks should be tested by the "Rattler wear test" as prescribed by the American Society for Testing Materials [3], [14]. In this test, the bricks are hammered and ground by the blows of large and small spheres in the rattler and the percentage loss of weight determines the quality of the material. However, no such test could be carried out on these bricks due to non-availability of this equipment.

4.1.3. Results

The results of compressive strength and water absorption tests on bricks obtained under normal conditions of firing indicate that first class bricks selected by visual inspection have given average compressive strength of 3600 lb per sq. in. and generally the strength is more than 3000 lb per sp. in.

The average strength of straight overburnt bricks is about 4900 lb per sq. in. and generally the strength is more than 4000 lb per sq. in.

The water absorption of first class bricks varied from 8.6 per cent to 12 per cent and that of overburnt bricks from 5.1 per cent to 8.13 per cent.

Results of compressive strength and water absorption tests carried out on first class and straight overburnt bricks produced by burning for extra periods, are given in Table 5 and shown in Plate II.

It would be seen that compressive strength of bricks increases by extra burning, the maximum strength being obtained at 12 hours extra burning when the average strength of straight overburnt bricks was 7293 lb per sq. in. and water absorption 5.82 per cent. Fairly good results are obtained in the case of first class and straight overburnt bricks by burning for 6 hours extra period, the strength obtained being 5619 and 6485 lb per sq. in. for first class and straight overburnt bricks respectively.

It would thus appear that extra burning for 6 to 12 hours is effective. It would be apparent that the maximum strength obtained in the kiln is not as high as that obtained in the laboratory and the period of firing is greater in the kiln than in the laboratory furnace. This variation is due to different conditions of burning in the two cases and also due to difference in the control of firing.

The period of firing to obtain the desired strength in bricks cannot be kept the same as may be sufficient in the laboratory furnace but laboratory results do give a fair indication about the suitability of soil for producing bricks of required quality.

4.2. Tests on Brick Metal

4.2.1. As stated before, by extra firing in the kiln, the percentage of mis-shapen straight overburnt bricks was fairly high in comparison to that obtained in an ordinary kiln.

4.2.2. It was proposed to use brick metal and brick grit produced by breaking these mis-shapen straight overburnt bricks in place of stone metal and stone grit both for consolidating as water-bound macadam and for surface painting work.

4.2.3. The brick metal was, therefore, got tested in the laboratory for (1) Los Angeles abrasion value and (2) aggregate crushing value, according to I.S. Specifications [15]. Water absorption was also determined.

4.2.4. The results indicated that the Los Angeles abrasion values of brick ballast vary from 30.45 to 40.7 per cent; the aggregate crushing values vary from 24 per cent to 31.65 per cent and the water absorption varies from 3.6 per cent to 7 per cent.

4.2.5. Effect of extra burning on these properties is not well marked because the selection of such bricks is based purely on appearance and in all compartments the same type of bricks might have been selected (mis-shapen straight overburnt bricks were produced in all compartments in varying percentages).

4.2.6. The results showed that brick ballast is quite suitable for use in place of stone metal for top coat work and also for painting.

5. USE OF BRICKS IN ROAD PAVEMENT

The bricks obtained from the kiln were used for laying an experimental pavement in mile 20 and $4\frac{1}{2}$ fgs. of mile 28 of Lucknow-Sultanpur Road.

5.1. Stacking of Bricks

5.1.1. In mile 20, the bricks as obtained from various rows of kiln fired for different intervals were stacked separately for being laid in separate lengths. Ordinary moulded and pug moulded bricks were also separately stacked along the road side. In mile 28, however, it was decided to select well shaped, uniform colour bricks for being laid in a particular length and to lay the remaining bricks in another length. No consideration of firing for different intervals was made here.

5.2. Structure of Brick Pavement

5.2.1. Preparation of the kankar Surface (foundation course)

Brick pavements are laid on various types of foundations both flexible and rigid. When surfacing is done with bricks on an old surface as a foundation, any variation in the old surface may be corrected by levelling up with water-bound macadam or other suitable material. In the present case the existing road had kankar surface, the average thickness of kankar being $7\frac{1}{2}$ in. in mile 20 and 8 in. in mile 28. The kankar surface had some ruts and was not smooth. In order to provide a surface free of ruts and pot holes and having proper camber to lay brick pavement resurfacing of kankar was done (See Specifications, Appendix-B).

In order to observe the behaviour of brick pavement on freshly laid kankar surface as well as on the kankar surface

Photo 1.
A general view of the Bull's kiln under operation

Photo 2.
Measurement of temperature by optical pyrometer

Photo 3.
Laying of brick pavement in camber

Photo 4.
Laying of brick-on-edge in herring-bone bond

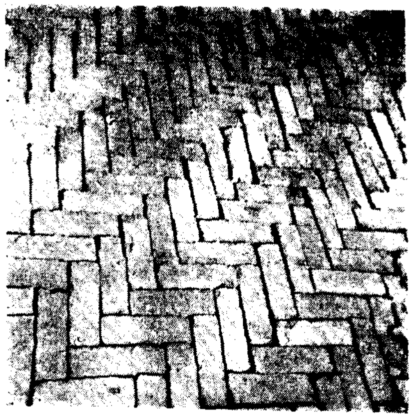

Photo 5.
The header-stretcher bond

Photo 6.
The completed pavement before opening to traffic

Photo 7.
The pavement under traffic (note the earth blinding on top)

Photo 8.
The brick pavement being laid in cement/sand mortar

Photo 9.
The brick pavement surface after primer coat has been given

Photo 10. **Application of sand-bitumen levelling course prior to painting**

Photo 11. **¾ in. premix carpet being laid on brick pavement surface**

Photo 12. **Rolling of premix carpet**

which has been subjected to traffic for sometime, it was decided to open some portion of the kankar surface to traffic before laying brick pavement. On the remaining portion, brick pavement was laid soon after compaction.

The kankar resurfacing was done in August-September 1957 in mile 20 and in November-December 1957 in mile 28.

5.2.2. Cushion or Bedding Course

Before laying bricks, locally available soil in the borrow-pits as well as fine Gomti sand were used in different lengths as a levelling or bedding course over the kankar surface which was not smooth and level throughout the length and also in filling the joints between bricks. Local earth or sand was also used in providing cushion in some lengths.

In mile 20, the thickness of levelling or bedding course provided was only sufficient to fill up the surface depressions and make the surface level excepting in a length where sufficient sand cushion (3 in. thick) was given. In mile 28, about 1 in.-1½ in. of earth cushion was provided over the kankar surface before laying bricks.

5.2.3. Brick Pavement

In order to observe the behaviour of brick pavement with flat bricks, as well as brick-on-edge, both types were adopted. Flat bricks were used in the first two furlong length of mile 20. In the remaining furlongs of mile 20 and 4½ fgs. of mile 28, brick-on-edge was adopted.

On an average, 350 bricks were used in laying flat brick pavement per mile for 12 ft width and 550 bricks were used in laying brick-on-edge pavement, the average size of brick being 9 in. × 4½ in. × 2¾ in.

The thickness of brick pavement provided was thus 3 in. and 4½ in. approximately as against 2¼ in. compacted thickness provided for stone metal and 3 in. compacted thickness provided for overburnt brick metal in modernisation work.

5.3. Specifications of Brick Pavement

5.3.1. The following three types of bonds were adopted :

(i) Herring-bone bond.

(ii) Header-stretcher bond,

(iii) Stretcher (transverse) bond.

Details of these bonds are shown in Plate III.

5.3.2. The following types of pavements were laid in mile 20:

(a) Flat brick pavement with local earth or sand for filling joints and for levelling course.

(b) Brick-on-edge with local earth or sand for filling joints and for levelling course.

(c) Brick-on-edge laid in cement-sand mortar over overburnt brick metal water-bound macadam surface.

(d) Brick-on-edge laid in cement/sand mortar as above, with $1\frac{1}{2}$ in. concrete topping over the brick surface.

5.3.3. In mile 28, brick-on-edge pavement was laid as per details in (b) above with the difference that 1 in.–$1\frac{1}{2}$ in. cushion of local earth was provided over the kankar surface before laying bricks.

5.3.4. The pavement in mile 20 was laid during September-October 1957 excepting brick pavement in cement-sand mortar which was laid during March-April 1958. In mile 28, the pavement was laid during January–February 1958.

5.3.5. Detailed specifications of the brick pavement are given in Appendix B. The different lengths in which these specifications have been used are given in Table 6 and shown in Plate IV which also shows typical cross sections of the brick pavement.

5.4. Traffic Intensity

5.4.1. Traffic census was taken out in mile 21 of this road during February 1958. The following figures of traffic intensity were evaluated :

7 days Traffic Census : Width of road 12 ft.

(a) Average traffic intensity per day

		Tons
(i) Power driven vehicles	...	687
(ii) Animal driven vehicles	...	77
Total	...	764

(b) Maximum traffic intensity per day

		Tons
(i) Power driven vehicles	...	800.5
(ii) Animal driven vehicles	...	95.0
Total	...	895.5

5.4.2. As there is no other secondary road meeting this road upto mile 35, this traffic intensity can be considered for both the miles 20 and 28.

5.5. Behaviour of Brick Pavement—Mile 20

5.5.1. After blinding the brick surface with sand or local earth, whichever was used in filling the joints, the surface was soon opened to traffic.

5.5.2. The traffic census data given in para 5.4.1 reveals that power driven vehicles predominate over animal driven ones, the latter being 10–11 per cent of the total intensity. Due to pneumatic tyred traffic the blinding over the brick pavement blew away soon, leaving the surface naked. A gang of three men was, therefore, kept in a mile to blind the surface with local earth continuously. It was observed that some bricks in portions of brick pavement began to be crushed under traffic causing depressions and irregularities in the surface, other bricks remaining intact. The edges of bricks also got rounded by traffic.

5.5.3. By constant blinding and passing traffic, the joints between bricks got compacted by local earth and there was no movement of bricks under traffic. In fact, when attempt was made to remove crushed bricks, difficulty was experienced.

No blinding was done on the brick pavement laid in cement mortar.

5.5.4. In portions where sand was used in the joints, it was seen that due to movement of bricks under traffic, the sand settled down forcing the bricks up and thus leaving them above the general level of road. These bricks got severe hammering under traffic and got damaged.

5.5.5. The general behaviour of different experimental portions of brick-pavement in mile 20 was as follows during May 1958 (before putting surface treatment).

(i) The portions laid in transverse bond behaved better than those laid in other bonds.

(ii) In header-stretcher bond, header bricks got more damaged. The surface in some cases became very uneven.

(iii) There was no difference in behaviour of ordinary moulded and pugged bricks.

(iv) The bricks burnt for 6 hours extra interval, behaved better than bricks burnt under normal conditions. This is also revealed by the compressive strength obtained for such bricks.

(v) The brick surface laid in cement-sand mortar was behaving much better than other surfaces. There were no depressions excepting crushing of few bricks here and there. The edges of bricks got rounded as cement—sand mortar sank a little in the joints leaving a slight gap.

(vi) The concrete overlay over the brick surface was also behaving satisfactorily for some time. There was only one transverse crack in the middle. Later on, however, a few pot holes were formed in the concrete surface near the end of experimental length. It appeared that there was not much bond between the concrete overlay and brick surface as well as mortar in joints as the concrete was put sometime after the brick pavement had been laid.

The pot holes were patched up by bitumen premix. It, however, appears that this surface may not stand for very long.

(vii) It appears that the surface of bricks laid over freshly consolidated kankar surface had worn out more than in the other case in the beginning but later on the surface was not so irregular. Some portion in this length has been left over unpainted for further observation.

5.5.6. Brick Pavement on Sand Cushion—Mile 20

This length was laid during June 1958 as per specifications given in Appendix B and was soon opened to traffic. The surface has been behaving satisfactorily and bricks have not been crushed, excepting that bricks have settled down under traffic particularly at places some distance away from the edges of the road.

The effectiveness of sand cushion in reducing the crushing or wearing out of bricks is apparent. However, under traffic and vibration, the 3 in. sand cushion appears to get compacted gradually and settle in the portions where traffic may be passing. In such cases, the bricks can be relaid to camber after the sand layer has been compacted.

5.5.7. Mile 28

In this mile, as stated before, bricks were selected and then laid. Earth cushion was also provided. Constant blinding of surface was resorted to as in mile 20. The observation of the behaviour indicated that:

(i) By provision of earth cushion, the percentage of crushed or worn out bricks was reduced considerably, although the intensity of traffic in this mile was the same as in mile 20.

The wearing out of bricks was also not severe although the edges of bricks got rounded. The surface was quite satisfactory.

(ii) Selection of good and uniform quality bricks was also responsible for better behaviour as in those portions where inferior quality bricks were put in, bricks got crushed and worn out soon after they were opened to traffic. In these damaged portions, bitumen premix was used to fill up the patches to the level of pavement and the surface behaved satisfactorily thereafter and further damage to brick surface was not observed.

(iii) Some portions of brick pavement in this mile are still unsurfaced and are taking traffic satisfactorily. No blinding is now being carried out.

6. BITUMINOUS TREATMENT ON BRICK PAVEMENT

6.1. After noting the behaviour of brick pavements (both with flat-brick and brick-on-edge) laid on kanker surface in miles 20 and 28 it was decided to provide a bituminous coat over some of these lengths with different specifications with the following objects in view:

(i) To observe the behaviour of different specifications of surface dressing as regards adhesion and durability, using tar or bitumen as binder and different products as primer.

(ii) To observe the behaviour of premix carpet.

(iii) To provide a bituminous coat over those portions of brick pavement which had shown wearing leading to irregularities in the surface and to observe their effectiveness as regards further stoppage or reduction of wearing.

6.2. Specifications

The following specifications were adopted:

6.2.1. Surface Painting First coat with Tar or Mexphalt and Stone Grit

In mile 20, before painting, a primer was applied on the brick surface. For this Shell primer, Road Tar R.T.-2 and Mexphalt 80/100 (primed with 40 per cent kerosene oil) were used in different lengths. As the brick surface was not uniform, a levelling course consisting of coarse sand mixed with 8 per cent Mexphalt 80/100 was applied to fill in depressions and was rolled. The surface was then painted, as usual. Painting was done both on flat brick and brick-on-edge surfaces.

6.2.2. Surface Painting in Mile 28

In this mile, R.T-1 (Road Tar) was used for priming the surface. As the surface of brick pavement was fairly smooth, no levelling course was provided. The surface painting was done with the usual specifications using R.T.-2 and grit. Some portions in this mile were left unpainted for observation and further trials.

6.2.3. $\frac{3}{4}$ in. Premix Carpet

Before laying premix carpet, the brick surface was primed by Shell primer or Mexphalt 80/100 mixed with 40 per cent kerosene oil. Premix carpet consisting of Mexphalt and stone grit was then laid and rolled.

The carpet was laid both over flat brick and brick-on-edge surfaces in mile 20. Detailed specifications are given in Appendix B.

6.2.4. Behaviour of Bituminous Coat

By surface painting, further wearing out of the surface has been stopped altogether.

The grit has been blown off from portions of the surface exposing the bricks. It appears that size of the grit usually used for first coat painting may not be suitable for painting on brick surface.

Grazing has been observed on the flat brick surface along few joints specially near the edges indicating movement of bricks. This may be due to the fact that joints in bricks at edges might not have been fully compacted by traffic.

The premix carpet has given quite satisfactory and smooth surface and is adhering well. The use of Mexphalt 80/100 mixed

with 40 per cent kerosene oil as primer is also quite effective and economical.

The portions in which Shell primer has been used are more exposed than those in which R.T.-2 has been used as a primer. This applies to those portions where Road Tar R.T.-2 has been used for surface painting.

In those portions where Mexphalt 80/100 has been used for surface painting, the surface has come out better than those where road tar was used. The surface has been exposed at some places here also but to a much lesser extent as compared to tar painted surface. This is also confirmed by laboratory tests to determine affinity of tar and bitumen with brick metal. The results given in Table 7 indicate that adhesion with Mexphalt is better than with tar.

The brick surface in mile 28 is more exposed than in mile 20. This may be due to the following factors :

(i) Use of better and harder quality bricks in mile 28 on which grit might not have adhered effectively. The bricks in this mile were not crushed and were standing well under the traffic.

(ii) In mile 28, R.T.-1 was used as primer over brick surface which may not have proved as effective as other primers.

(iii) No levelling course of sand-bitumen premix was laid in mile 28, as there were not many depressions and irregularities in the surface.

This also points out to the necessity of using smaller size grit for painting.

7. BRICK PAVEMENT WITH BITUMINOUS JOINT FILLER

7.1. The following experimental lengths of brick pavement using bituminous joint filler were also laid during June 1958 (in mile 20):

(i) Brick-on-edge on earth cushion with joints filled in with bituminous filler (Mexphalt 80/100).

(ii) Brick-on-edge over sand-bitumen mastic, with joints filled in with bituminous filler (the sand-bitumen mastic being laid after the kankar surface had been primed with a bituminous primer).

7.2. Detailed specifications are given in Appendix B and shown in Plate IV.

7.3. Behaviour of Brick Pavement with Joints filled with Bitumen

7.3.1. The brick pavement laid over earth cushion with joints filled with bitumen is behaving better than the pavement laid over sand-bitumen mastic. Some bricks laid over bitumen mastic were crushed under traffic after some time. It appears that earth cushion plays an important part in reducing the crushing of bricks. As the bitumen mastic was compacted by roller, fairly hard surface was obtained which tended to crush the bricks under traffic.

The mastic should have been very lightly rolled before laying brick pavement as is specified in American specifications. [16]

7.3.2. Some bitumen was left over the top of the bricks while pouring into the joints and this could not be removed in spite of the use of lime-wash on the top surface of the bricks. This bitumen was allowed to remain there and it has not given any trouble to traffic. Actually it has helped in preventing wearing out of the brick surface.

7.3.3. It was also seen that although bitumen was filled in to the joints completely in the beginning, it sank later on leaving slight gaps on the top.

It is proposed to fill in the joints by fine sand.

7.3.4. The quantity of bitumen required for filling the joints was excessive which increased the cost. This was due to bricks not being of uniform size and shape.

8. COST OF BRICK PAVEMENT

8.1. The rates of labour and cost of materials (excluding the cost of bricks) for laying brick pavement in different specifications and for providing bituminous coat over the brick surface are given in Appendix C.

The costs of providing pavement (taking the cost of bricks as Rs 45 per thousand) with bituminous coat on top for some of the specifications are also given in Appendix D.

8.2. It may be noted that these costs are based on experimental stretches laid in short lengths and it can be safely

presumed that the costs will be lower if work on a large scale is undertaken. Observations regarding costs are as follows:

(1) The cost of flat brick pavement (in transverse bond) with local earth for filling joints and surface painting with Mexphalt (without any levelling course of bitumen-sand on brick surface) compares well with the cost of modernisation of a road by providing 4½ in. of overburnt brick metal water-bound macadam with surface painting on top. The cost of this type of flat brick pavement is much less than the cost of modernisation by using stone metal.

(2) The cost of flat brick pavement over 3 in. sand cushion is less than the cost of providing 6 in. kankar road.

Renewal of kankar surface is required every 2 or 3 years while in the case of bricks, this may not be required. Surface painting on top will provide a wear resisting coat if traffic is high.

(3) Where the thickness of kankar coat is 4 in.-5 in. and the road is to be modernised, it will be necessary to provide a coat of kankar before modernisation is done. In its place, brick-on-edge pavement with 1 in.-1½ in. earth or sand cushion laid in transverse bond and surface painting on top can be used effectively and cheaply too.

(4) In constructing new village and other district roads carrying medium intensity of traffic, brick-on-edge over 2 in.-3 in. sand cushion may be provided. If the soil is already sandy, the sand cushion can be omitted.

(5) Where the thickness of existing kankar or other type of crust is already 6 in. or more, and where modernisation with two coat surface dressing on stone water-bound surface is provided, it is suggested that ¾ in. premix carpet over brick-on-edge pavement in transverse bond laid with 1 in. - 1½ in. earth cushion may be adopted, as there is not much difference in cost and greater thickness of pavement is provided. Besides, the ¾ in. premix carpet will give a smooth riding surface.

(6) Where a road is to be constructed in waterlogged areas or where widening is to be done, flat brick course followed by a layer of 1½ in. sand and then brick-on-edge pavement may be suitable. If there are no chances of settlement, surface painting may be given shortly after laying the brick. If, however, the pavement settles, the bricks can be relaid and surface treatment given.

The cost of these specifications compares favourably with the cost of black-top surface with 8 in. crust.

9. CONCLUSIONS

The behaviour of the brick pavement laid in different specifications indicates that:

(1) Brick-on-edge pavement laid with 1 in. to $1\frac{1}{2}$ in. earth cushion over an existing road can take traffic upto 800 tons/day satisfactorily.

A coat of surface dressing with Mexphalt and grit gives further protection to the brick surface and further wearing or crushing of bricks, if any, is stopped. In order to provide better and smooth surface, $\frac{3}{4}$ in. premix carpet is recommended.

(2) Flat brick pavement has also given satisfactory results and it is expected that with 1 in. - $1\frac{1}{2}$ in. cushion over the existing road surface this pavement should also behave satisfactorily.

Here also provision of surface dressing will help in stopping the wear and tear of surface. $\frac{3}{4}$ in. premix carpet will provide a much better and smoother surface.

This pavement is economical, as lesser number of bricks and fewer joints are involved. The cost of flat brick pavement compares favourably with the cost of modernisation by using overburnt brick metal or stone metal.

(3) For surface dressing over brick pavement, the usual size ($\frac{1}{4}$ in. - $\frac{3}{8}$ in.) grit used for first coat painting does not adhere well to the surface and it is considered that smaller size grit may give better results.

It is proposed to try specifications including the use of coarse sand for surface painting over brick pavement.

(4) In case of brick pavement, softer bricks get crushed under traffic in the beginning forming depressions. If these patches are covered by bituminous premix to the level of the pavement, further wearing of the surface is prevented.

(5) Brick pavement laid in cement-sand mortar has behaved fairly satisfactorily but the cost is high and it may need a bituminous coat on top later on. The behaviour of concrete overlay is not satisfactory and if it is desired to use these specifications, it is suggested that the joints between bricks should be wider and the concrete should be laid simultaneously in the joints and at the top to form a composite section.

(6) The brick pavement with joints filled in with bituminous filler is expensive to lay and cannot be recommended for adoption in place of other specifications of brick pavement.

(7) As subgrade conditions in U.P. and other States will not be as bad as in Holland and other countries (except in waterlogged areas), experimental stretches of brick pavement laid directly on earth subgrade may be successful and can be tried. In waterlogged areas the brick pavement offers a good solution as any subsidence or settlement in the surface can be rectified easily. A sand cushion may be needed in such locations.

(8) It is possible to burn fairly strong bricks in the ordinary Bull's kiln if properties of brick clay are predetermined by means of tests to find its suitability for producing better quality bricks and if control is exercised on its mechanical composition and properties and also in the firing of bricks in the kiln.

It has been seen that extra firing can improve the quality of bricks to some extent.

The mis-shapen overburnt bricks obtained in the kiln can be used for producing brick metal for use in road construction in place of stone metal. The former will be cheaper.

(9) For withstanding the wear and tear by heavy iron-tyred traffic of bullock carts, either vitrified bricks should be obtained and used or bricks possessing fairly high strength (7000-8000 lb. per sq.in.) should be laid with some sand/earth cushion and bituminous coat provided on the top later on, if necessary.

The experimental brick pavement laid on Lucknow-Sultanpur Road has been taking both types of traffic, viz., animal driven and power driven. Although the animal driven traffic which consists of bullock carts is not heavy, still it is seen that pavement having fairly high compressive strength bricks has taken the bullock-cart traffic satisfactorily.

This needs further experimentation and it is suggested that straight overburnt bricks produced from an ordinary kiln may be laid over sand cushion in village and other district roads.

(10) Possibilities of establishing Hoffmann type kiln for producing high compressive strength bricks may be explored at some selected places from where bricks may be carted and used on various roads, under different conditions of subgrade, etc. If successful, many factories will spring up throughout the country to provide bricks for dustless low cost roads.

It is, however, necessary that brick kiln owners and the authorities using these bricks should co-operate in making this project a success. It may not be possible for Govt. Departments or local authorities to invest money in establishing such a kiln.

Investigations into the possibilities of producing high compressive strength bricks from other types of kilns like Down Draft type may also be carried out as this type of kiln may be cheaper to set up than a Hoffmann type kiln.

This problem is already under consideration of the U.P.-P.W.D. Research Institute and it is hoped that trials for producing high quality bricks from such kilns may be carried out soon for laying brick pavements.

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Appendix A

Details of Bull 's Kiln

Capacity of kiln	= 6 lakh bricks per round.
Overall outer length	= 223 ft
Inner length (straight portion)	= 157 ft
Width of trench	= 24 ft
Number of compartments (1st round)	= 20
Number of compartments (2nd round)	= 19
Number of rows (combustion chambers) in a compartment	= 8

Depth of trench : 7 ft 3 in. (4 ft below ground level and 3 ft 3 in. above ground level).

Height of chimneys - Outer chimney 31 ft
Inner chimney 30 ft

Land acquired:

Permanent 2.5 acres
Temporary 3.0 acres

Orientation of kiln : East-West.

Appendix

SPECIFICATIONS OF BRICK PAVEMENT

Specification No. 1 (a)

Laying Flat Bricks over Kankar Surface

Existing kankar surface was scarified to an average depth 2 in. and the resulting surface brought to camber. The surfa was cleaned and charri from the excavated mass picked ou Fresh kankar at the rate of 1500 cu. ft. per furlong (for the 12 wide road) was first spread and then the bigger pieces of kank from the salvaged mass were laid and consolidated using a pow roller (8 tons). As the kankar available in this area is of a extremely hard nature the use of power roller was adopted. Tl surface was finished to a camber of 1 in 60. The surface w then opened to traffic for a period not exceeding one month.

When laying brick pavement the surface was first covere and levelled with the local fine sand. The bricks were then lai flat to proper camber over this sand covering as close together a possible keeping the size of joints to a minimum. The bricks wer laid in two bonds, viz., in header and stretcher bond and i herring-bone bond. Details of the bonds are shown in Plate II The joints between bricks were filled up with the local fine sanc The surface was also blinded with this sand. Before opening th surface to traffic, the earthen shoulders on both sides were pu level to the brick surface in order to protect the edges o the brick pavement and also to resist the movement of brick at the sides under traffic.

Specification No. 1 (b)

This specification is almost the same as specification a 1 (a) except that in place of local fine sand, locally available eart was used for levelling, joint filling and blinding.

Specification No. 2 (a)

Laying Brick-on-edge over Kankar Surface

Bricks were laid over the kankar surface similar to the specification 1 (b) except that here the bricks were laid on edge.

In addition to the two types of bonds as in 1 (a), bricks were also laid transversely, with the longer dimension perpendicular to the centre line of the road. Details of this bond are shown in Plate III.

Specification No. 2 (b)

Here the specification was similar to that in 2 (a) except that local sand was used instead of local earth in levelling, joint filling and blinding.

Specification No. 3

Laying Brick-on-edge over 3 in. Sand Cushion (over kankar surface)

On the surface of kankar as prepared in specification 1 (a), sand was placed and compacted by hand ramming. On both edges of the road surface, bricks-on-end were placed to retain the sand. The bricks-on-end were also supported by putting earth shoulders on outer sides. On the sand cushion, bricks-on-edge were placed using sand for filling joints. The bricks were laid transversely as described in 2 (a).

Specification No. 4

Laying brick-on-edge over Kankar Surface using Joint Filler

Bricks-on-edge were laid as per specification 2 (a) except that the joints were not filled with earth, and the thickness of earth cushion below the bricks was kept 1 in.-1½ in. The bricks were laid transversely. Mexphalt 80/100 was used for filling the joints. The brick surface was first given a wash of separating agent, viz., lime in water, taking care that the solution did not get into the joints. The joints were then filled up by pouring the hot Mexphalt 80/100 through a suitable container, making sure that the filler reached the bottom of the joints.

Specification No. 5

Brick-on-edge over Sand-bitumen Mastic Bed using Joint Filler

Over the kankar surface, a mastic bed ½ in. thick was laid using clean dry coarse sand and 8 per cent Mexphalt 80/100. Prior to laying the mastic, a coat of bituminous primer of Mexphalt 80/100 mixed with 40 per cent kerosene oil was applied over the kankar surface at the rate of 25 lb per sq. ft. The mastic surface was rolled with a roller to bring the surface to proper grade and camber. Bricks-on-edge were then laid keeping the joints as narrow as possible. The bricks were laid transversely. The surface was again rolled and the joints were then filled up with Mexphalt 80/100 as per specification No. 4.

Specification No. 6

Brick-on-edge over Freshly Consolidated Kankar Surface

The kankar surface was prepared as per specification 1 (a). No traffic was allowed over it and bricks-on-edge were directly laid over this surface. The joint filling and surface blinding was done with earth. Final rolling was then done over this surface with sprinkling of water. The surface was examined after rolling and any defects on the finished surface rectified.

The bricks were laid in header-stretcher, herring-bone and transverse bonds.

Specification No. 7 (a)

Bricks-on-edge over 1 : 3 Cement Sand Mortar laid over Brick Ballast Surface using 1 : 3 Mortar as Joint Filler

A 3 in. coat of water-bound overburnt brick ballast was given over the existing surface according to usual specifications. In consolidation fine sand was used. After the surface had set, it was cleaned using wire brushes. Cement-coarse sand mortar (1 : 3) was spread to an average thickness of ¼ in. to 3/3 in. Bricks-on-edge were then laid over this mortar using 1 : 3 mortar for filling the joints. No mortar was put over the bricks and the mortar in joints was flush with the bricks. The bricks were laid transversely. The surface was cured for 28 days before opening to traffic.

Specification No. 7 (b)

Cement Concrete Topping over Brick-on-edge laid in Cement Sand Mortar

In these specifications, bricks-on-edge were laid in cement sand mortar similar to specification 7 (a). Over the brick surface, a layer of 1 : 2 : 4 cement concrete 1½ in. thick was laid and compacted, after removing the loose material in joints and over the brick surface. The concrete was cured for 28 days by ponding prior to opening for traffic.

SPECIFICATIONS FOR BITUMINOUS COAT

(a) Specifications for ¾ in. Premix Carpet

Before laying premix carpet, the brick surface with flat bricks or bricks-on-edge was cleaned of all loose earth from the top and the joints. A priming coat of bituminous material was then applied over the brick surface at the rate of 25 lb per 100 sq. ft. The following primers were used in different lengths:

(1) Shell primer No. 2.

(2) Mexphalt 80/100 + 40 per cent kerosene oil.

Note:

The primer at serial 2 was prepared at the site by heating Mexphalt 80/100 slightly and then pouring it in kerosene oil, stirring the whole vigorously and continuously until a homogeneous mixture was obtained.

Premix was prepared by hand mixing stone grit with Mexphalt 80/100 at the rate of 3½ lb per cu. ft. of grit, the grit being used at the rate of 12 cu. ft. per 100 sq. ft. of surface. The premix was then laid over the primed surface to ¾ in. thickness and rolled by a power roller, taking care to provide a camber of 1 in 60 on the finished surface.

The carpet was blinded with coarse sand to fill up the voids and to give smooth riding surface.

(b) Specifications for surface painting

The brick surface (with flat bricks or brick-on-edge) was cleared of all dust and loose material.

A bituminous primer was then applied to the surface at the rate of ·5 lb/sq. ft. In addition to Shell Primer No. 2 and Mexphalt 80/100+40 per cent kerosene oil, Road Tars R.T.-1 and R.T.-2 were also used as primers in different lengths.

After priming, a thin levelling coat of sand bitumen mastic (8 per cent of Mexphalt 80/100 mixed with coarse sand) was laid over the brick surface to level up the depressions and irregularities in the surface. It was laid to camber by a wooden batter which was moved over the surface manually and then rolled. A minimum quantity of mastic was used just to fill up the depressions to the level of those bricks which had not been abraded.

Road Tar R. T.-2 or Mexphalt 80/100 (as the case may be) was then spread over the surface at the rate of 40 to 45 lb/100 sq. ft. by means of a rose can to cover the whole surface. Stone grit (½ in. size) was then spread over the surface @ 4.4 cu. ft. per 100 sq. ft. and it was rolled as usual.

The surface was then opened to traffic.

Appendix C

Abstract of analysis of rates for laying brick pavement

	Details of pavement		Rate/100 sq. ft. for labour only Rs
1.	Pavement with flat bricks using earth for levelling and filling joints	...	3.30
2.	Pavement with flat bricks but using sand (including cost of sand)	...	4.30
3.	Pavement with brick-on-edge using earth for levelling and filling joints	...	4.40
4.	Pavement with brick-on-edge using sand (including cost of sand)	...	5.58
5.	Brick-on-edge pavement over 3 in. 1 : 3 sand cushion	...	9.58
6.	Pavement with brick-on-edge in cement-sand mortar	...	48.50
7.	**Bituminous Treatment:**		
	(a) Cost of priming	...	4.00
	(b) Cost of patching with sand-bitumen	...	5.00
8.	Painting 1st coat with R. T.-2 over patched up surface	...	16.00
9.	Painting 1st coat with mexphalt over patched up surface	...	17.00
10.	¾ in. premix carpet with priming coat	...	26.00
11.	Filling joints with bituminous filler	...	39.00
12.	Paving bricks-on-edge over mastic and joints filled with bituminous filler (application of primer before laying mastic)	...	64.00

Note:

(1) For brick-on-edge pavement, bricks required are 550 and for flat brick pavement, bricks required are 350 per 100 sq. ft.

(2) The above rates are for header-stretcher and herring-bone bonds. For transverse bond, the rate is 10 per cent less. The rate for items 1, 2 and 3 is thus Rs 3, Rs 4 and Rs 4 respectively per 100 sq. ft. and for item 4, it is Rs 5.00 per 100 sq. ft. and for item 5, it is Rs 9 per 100 sq. ft.

Appendix D

Cost of different types of road pavement including brick pavement

Sl. No.	Specifications	Cost per mile (12 ft wide road.) Rs
1. (a)	Brick-on-edge pavement-sand filling in joints-surface painting (Mexphalt) ...	36,340
(b)	,, ,, ,, ,, (without levelling course) ...	33,170
(c)	,, ,, ,, ,, earth filling in joints	32,425
(d)	,, ,, (transverse bond) ,, ,, ...	32,170
(e)	,, ,, ,, ,, ¾ in. premix carpet ...	35,370
2. (a)	Flat brick pavement-local earth in joints-transverse bond without levelling course-surface painting Mexphalt ...	25,635
(b)	,, ,, ,, ,, ,, premix carpet ...	28,354
3.	Brick-on-edge over 3 in. sand cushion (transverse bond) ...	22,000
4.	Flat Brick ,, ,, ,, ,, ,, ...	15,400
5.	Brick-on-edge ,, 2 in. sand cushion ,, ,, ...	21,400
6.	Brick-on-edge over 1½ in. sand laid over flat bricks	33,050
7. (a)	Modernisation over stone metal (2¼ in.) W.B.M.-surface painting one coat ...	29,078
(b)	,, ,, ,, ,, two coats ...	34,578
8.	Modernisation over brick metal (3 in.) surface-painting one coat ...	25,800
9.	Kankar (2 coats) 6 in. ...	16,300
10.	,, (3 coats) 9 in. ...	24,280
11.	Modernisation (2¼ in. stone metal) over 6 in. kankar-surface painting-one coat ...	45,375
12.	Surface painting by Mexphalt and priming (one coat) ...	13,305

Sl. No.	Specifications	Cost per mile in Rupees (12 ft wide Rd.) Rs
13.	Surface Painting 2 coat ...	5,500
14.	¾ in. premix carpet with priming ...	16,500
15.	Resurfacing with kankar (3 in.) ...	7,980

Note :

1. The above cost includes the cost of renewal of patries (shoulders).
2. The following rates have been considered :
   - (i) Cost of bricks Rs 45 per thousand.
   - (ii) Cost of renewal of patries Rs 450 to 800 (depending upon the thickness of pavement).
   - (iii) Cost of stone metal Rs 100 per 100 cu. ft.
   - (iv) Rate of stone consolidation Rs 11 per 100 cu. ft.
   - (v) Cost of brick ballast Rs 50 per 100 cu. ft.
   - (vi) Cost of brick metal consolation Rs 7 per 100 cu. ft.
   - (vii) Cost of brick edging Rs 800 per mile.
   - (viii) Cost of kankar Rs 25 per 100 cu. ft.

TABLE 1

Showing physical characteristics of soils from Lucknow-Sultanpur Road

Serial No.	Lab. No.	Location	Liquid limit	Plastic limit	Plasticity index	Sand retained on 200 mesh	Sulphate content	Particle size analysis		
								Clay %	Silt %	Sand %
1	734	Mile 22 fg. 4—north	26.9	22.3	4.6	3.1	Nil			
2	735	Mile 22 fg. 4—south	Non-plastic			13.1	,,			
3	756	South Road corner mile 22 fg. 5	23.8	15.7	8.1	—	,,			
4	757	Road central pit	22.7	16.5	6.2	—	,,			
5	758	North—east corner	24.4	18.1	6.3	—	,,			
6	759	Field south	27.9	19.2	8.7	—	,,			
7	760	Mile 22 fg. 5—field north	30.6	20.3	10.3	1.07	,,			
8	778	Mile 22 fg. 5—centre	33.2	20.8	12.4	1.12	,,	22	36	42
9	779	Mile 22 fg. 5—south	36.8	24.0	12.8	0.59	,,	26	26	48
10	780	Mile 23—south	34.1	21.6	12.5	0.14	,,	19	31	50
11	781	Mile 23—north	30.0	18.0	12.0	0.95	,,	21	35	44

TABLE 2

Showing properties of laboratory burnt bricks

Serial No.	Laboratory sample No.	Burning details	Average compressive Strength lb per sq. in.	24 hours Water absorption per cent	Remarks
1	760	5 hours at 950°C	7,496	8.35	Suitable
2	778	5 hours at 950°C	5,600	12.65	Suitable
3	779	5 hours at 950°C	8,243	7.6	Suitable
4	779	7 hours at 950°C	10,523	5.9	Suitable
5	780	5 hours at 950°C	5,409	15.29	
6	781	5 hours at 950°C	4,594	12.23	

N.B.—Laboratory bricks 4.6 in. × 2 in. × 1.5 in. were moulded at 10 per cent less than the liquid limit moisture content of each soil. These bricks were first dried in shade for one week and finally in an oven at 50° to 60°C to constant weight. These bricks were then burnt in a laboratory muffle furnace at 950°C for 5 hours and 7 hours as mentioned above and then tested.

TABLE 3

Showing typical temperatures obtained in the kiln chamber in Holland

Day	1st	2nd	3rd	4th	5th	6th
Temperature °C.	50°C (Loading)	60°C (Loading)	100°	100-300	300-500	500-700
Day	7th	8th	9th	10th	11th	12th
Temperature °C.	700-900	900-1000	1000-1050	1060	1070	1080
Day	13th	14th	15th	16th	17th	18th
Temperature °C.	1090	1100	1100-1050	1050-1000	1000-800	800-650
Day	19th	20th	21st	22nd	23rd	24th
Temperature °C.	650-450	450-300	300-150	150-50	50 (Unloading)	50 (Unloading)

TABLE 4

Details of consumption of slack coal

Specifications of burning	Consumption of coal (tons)	Bricks burnt Nos.	Rate of consumption/lakh of bricks
1st Round of Firing			
1. Normal burning ...	115.60	4,65,000	25.0
2. Normal + ½ hour extra ...	2.70	10,600	25.5
3. Normal + 1½ hours extra ...	2.80	10,600	26.4
4. Normal + 6 hours extra ...	4.10	14,150	29.0
5. Normal + 9 hours extra ...	4.60	14,150	32.5
6. Normal + 12 hours extra ...	19.80	56,700	35.0
2nd and Part of 3rd Round of Firing			
1. Normal burning ...	36.30	2,27,200	16.0
2. Normal + 3 hours extra ...	2.80	14,900	19.0
3. Normal + 6 hours extra ...	56.30	2,73,100	20.5
4. Normal + 9 hours extra ...	3.40	14,900	22.5
5. Normal + 12 hours extra ...	7.40	29,800	24.8
6. Normal + 18 hours extra ...	3.20	11,200	28.4
7. Normal + 24 hours extra ...	3.70	11,200	33.0
8. Normal + 36 hours extra ...	4.60	11,200	41.7
9. Normal + 48 hours extra ...	5.00	14,900	44.7
10. One week burning ...	8.00	14,900	53.7
Total ...	280.30	11,90,500	23.6

TABLE 5

Showing results of tests on compressive strength and water absorption of bricks fired for extra period

Sl. No.	Extra period of firing in hours	Type of bricks: First class — Compressive strength lb per sq. in.	Type of bricks: First class — Water absorption per cent	Type of bricks: Straight overburnt — Compressive strength lb per sq. in.	Type of bricks: Straight overburnt — Water absorption per cent
1	½	4263	10.6	5417	7.7
2	1	4886	9.5	5898	7.8
3	1½	—	—	6053	7.0
4	3	5200	6.86	5805	7.3
5	6	5619	8.5	6485	7.48
6	9	5313	8.68	6697	5.13
7	12	4973	8.04	7293	5.82
8	24	4399	8.95	6639	5.5
9	36	5917	8.8	6414	4.9
10	48	5156	9.4	6821	5.8
11	7 days	4950	8.94	—	—

TABLE 6

Details of brick pavement in miles 20/28

	Length of brick pavement	Date of laying brick pavement	Bituminous treatment: Length of treatment	Bituminous treatment: Priming coat	Bituminous treatment: Levelling coat	Bituminous treatment: Surface dressing or premix carpet	Date of laying bituminous coat
	3		5	6	7	8	9
opened sand :							
extra ıd	330 ft		100 ft	Shell Primer 25 lb./100 sq. ft.	Bitumen mastic with coarse sand & 8 per cent Mexphalt 80/100	1st coat painting with R. T.-2	June 1958
extra	330 ft		550 ft	R. T.-2 25 lb. per 100 sq. ft.	do.	do.	
opened earth:							
using	350 ft		330 ft	do.	do.	do.	
using	50 ft		330 ft	Mexphalt 80/100 40 per cent kerosene oil 25 lb. per 100 sq.ft.		⅜ in. premix carpet (Mexphalt) 80/100 & stone grit	
using	50 ft						
s extra ond	160 ft						
surface ts with							
s extra ond	50 ft	September/ October, 1957	660 ft	R. T.-2 25 lb. per 100 sq.ft.	Bitumen mastic with coarse sand and 8 per cent Mexphalt 80/100	1st coat painting with R. T.-2	
s extra	98 ft						
s extra ond	232 ft						
surface h sand:							
using	300 ft						

TABLE 5

Showing results of tests on compressive strength and water absorption of bricks fired for extra period

Sl. No.	Extra period of firing in hours	Type of bricks: First class — Compressive strength lb per sq. in.	First class — Water absorption per cent	Straight overburnt — Compressive strength lb per sq. in.	Straight overburnt — Water absorption per cent
1	½	4263	10.6	5417	7.7
2	1	4886	9.5	5898	7.8
3	1½	—	—	6053	7.0
4	3	5200	6.86	5805	7.3
5	6	5619	8.5	6485	7.48
6	9	5313	8.68	6697	5.73
7	12	4973	8.04	7293	5.82
8	24	4399	8.95	6639	5.5
9	36	5917	8.8	6414	4.9
10	48	5156	9.4	6821	5.8
11	7 days	4950	8.94	—	—

TABLE 6

Details of brick pavement in miles 20/28

Specification No.	Details of pavement	Length of brick pavement	Date of laying brick pavement	Bituminous treatment: Length of treatment	Priming coat	Levelling coat	Surface dressing or premix carpet	Date of laying bituminous coat
1	2	3		5	6	7	8	9
	MILE 20							
A.	**Flat brick pavement over new kankar surface opened to traffic—cushion & filling joints with local sand :**							
(i)	With pugged bricks, usual + 6 hours extra burning using header & stretcher bond	330 ft		100 ft	Shell Primer 25 lb./100 sq. ft.	Bitumen mastic with coarse sand & 8 per cent Mexphalt 80/100	1st coat painting with R. T.-2	
(ii)	With pugged bricks, usual + 6 hours extra burning—using herring-bone bond	330 ft		550 ft	R. T.-2 25 lb. per 100 sq. ft.	do.	do.	
B.	**Flat brick pavement over new kankar surface opened to traffic—cushion and filling joints with local earth:**							
(i)	With pugged bricks, usual burning using herring-bone bond	350 ft		330 ft	do.	do.	do.	
(ii)	With pugged bricks, usual burning using header & stretcher bond	50 ft		330 ft	Mexphalt 80/100 40 per cent kerosene oil 25 lb. per 100 sq.ft.		⅜ in. premix carpet (Mexphalt) 80/100 & stone grit	June 1958
(iii)	With pugged bricks, usual burning using herring-bone bond	50 ft						
(iv)	With ordinary bricks, usual + 6 hours extra burning—using header & stretcher bond	160 ft						
C.	**Brick-on-edge pavement over new kankar surface opened to traffic—cushion and filling joints with local earth :**							
(i)	With ordinary bricks, usual + 6 hours extra burning—using header & stretcher bond	50 ft						
(ii)	With ordinary bricks, usual + 6 hours extra burning—using stretchers only	98 ft						
(iii)	With ordinary bricks, usual + 6 hours extra burning—using header & stretcher bond	232 ft	September/ October, 1957	660 ft	R. T.-2 25 lb. per 100 sq.ft.	Bitumen mastic with coarse sand and 8 per cent Mexphalt 80/100	1st coat painting with R. T.-2	
D.	**Brick-on-edge pavement over new kankar surface opened to traffic—cushion and filling joints with sand:**							
(i)	With ordinary Bricks, usual burning—using header & stretcher bond	300 ft						

Details of Brick Pavement in Miles 20/28—(*Contd.*)

Specification No.	Details of pavement	Length of brick pavement	Date of laying brick pavement	Bituminous Treatment: Length of treatment	Bituminous Treatment: Priming coat	Bituminous Treatment: Levelling coat	Bituminous Treatment: Surface dressing or premix carpet	Bituminous Treatment: Date of laying bituminous coat
1	2	3	4	5	6	7	8	9
E.	**Brick-on-edge pavement laid over kankar surface opened to traffic, cushion and filling joints with sand:**							
(i)	With ordinary bricks, usual + 6 hours extra burning—using herring-bone bond	318 ft	September/ October, 1957	330 ft	Shell Primer 25 lb. per 100 sq.ft.	—	$\frac{3}{4}$ in. premix carpet (Mexphalt 80/100 & stone grit)	June 1958
F.	**Brick-on-edge over kankar surface opened to traffic-cushion and filling joints with local sand :**							
(i)	With ordinary bricks, usual + 6 hours extra burning—using headers & stretchers	342 ft		330 ft	,,	Bitumen mastic with coarse sand and 8 per cent Mexphalt 80/100	1st coat painting with R.T. 2	
G.	**Brick-on-edge pavement over new kankar surface opened to traffic—cushion and filling joints with sand :**							
	With ordinary bricks, usual + 12 hours extra burning							
(i)	Headers and stretchers	72 ft						
(ii)	Herring bone bond	50 ft		325 ft	Shell Primer 25 lb. per 100 sq. ft.	Bitumen mastic with coarse sand and 8 per cent Mexphalt 80/100	1st coat painting with Mexphalt 80/100	
(iii)	Headers and stretchers	50 ft						
(iv)	Herring bone bond	50 ft						
(v)	Headers and stretchers	100 ft						
H.	**Brick-on-edge pavement over 3 in. layer of local sand (placed over kankar surface)**							
	Joints filled with local sand :	235 ft	June 1958					
I. (i)	**Brick on-edge pavement on 1 in.—1½ in. earth cushion placed over new kankar surface opened to traffic using joint filler :**			—	—	—	—	—
	With ordinary bricks, usual + 12 hours extra burning—using transverse bond	210 ft	,,	210 ft	—	—	Joints filled with Mexphalt 80/100	June 1958

Details of Brick Pavement in Miles 20/28—(*Concluded*)

Specification No.	Details of pavement	Length of brick pavement	Date of laying brick pavement	Bituminous treatment				
				Length of treatment	Priming coat	Levelling coat	Surface dressing or premix carpet	Date of laying bituminous coat
1	2	3	4	5	6	7	8	9
I. (ii)	**Brick-on-edge pavement laid over bitumen mastic ½ in. thick using joint filler:**							
	With ordinary bricks, usual + 6 hours extra burning using transverse bond	150 ft	June 1958	150 ft	Mexphalt 80/100 + 40 per cent kerosene oil over kankar surface	½ in. bitumen mastic (coarse sand & Mexphalt)	Joints filled with 80/100 Mexphalt	June 1958
J.	**Brick-on-edge pavement laid over freshly consolidated kankar surface (not opened to traffic), cushion and filling joints with earth:**							
(i)	With pugged bricks, usual burning—using stretcher bond (transverse)	100 ft	Sept./Oct. 1957	862 ft	Shell primer	Bitumen Mastic with coarse sand and 8 per cent mexphalt	1st coat painting with Mexphalt 80/100	
(ii)	With pugged bricks usual + 18 hours extra burning—using header and stretcher bond	100 ft						
(iii)	With pugged bricks—usual + 6 hours—using header & stretcher bond	200 ft						
(iv)	With pugged ordinary bricks—using stretchers only (transverse)	505 ft						
(v)	With pugged ordinary Herring-bone bond	85 ft		128 ft		(No treatment)		
(vi)	With pugged bricks—usual burning—using herring-bone bond	380 ft		281 ft	Shell primer	Bitumen mastic with coarse sand & 8 per cent Mexphalt	1st coat painting with Mexphalt 80/100	
K.	**Brick on-edge pavement with cement and coarse sand 1 : 3 over brick ballast consolidated with local *Gomti sand*:**							
(i)	Ordinary bricks, usual burning—using header & stretcher bonds	280 ft	March/April 1958	1½ in. cement concrete 1 : 2 : 4 laid on top		(No treatment)		
(ii)	Ordinary bricks—usual burning—using Herring bone bond	50 ft						
	MILE 28							
L.	**Brick-on-edge pavement laid over new kankar surface opened to traffic—with 1 in. earth-cushion and joint filling with local earth:**							
(i)	Using herring-bone bond	1580 ft	Jan./Feb. 1958	1250	R.T.-1	—	1st coat painting with Tar R.T.-2.	July 1958
(ii)	Using header & stretcher bond	220 ft		220		(No treatment)		
(iii)	Using tranverse bond	982 ft		—				

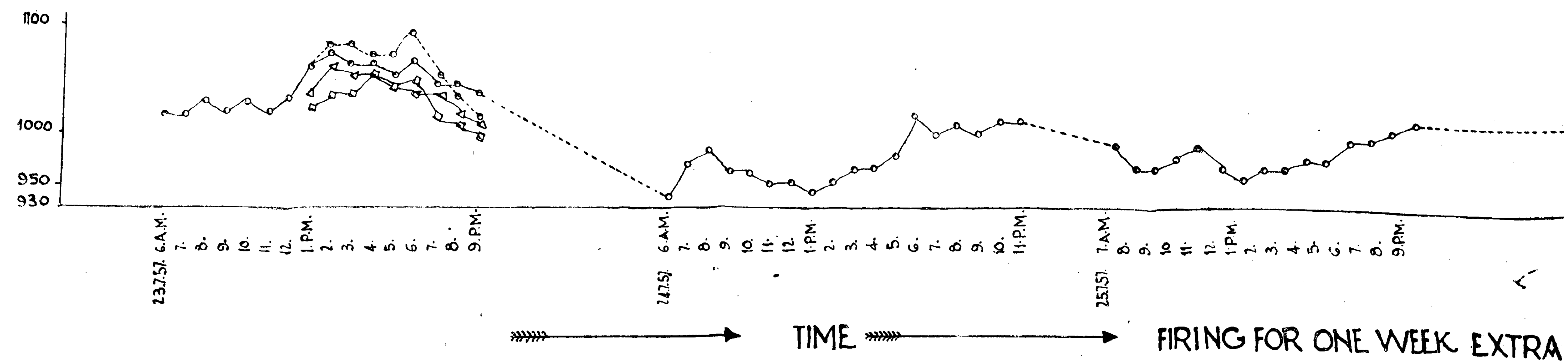
TABLE 7

Tests on affinity of brick metal produced from overburnt bricks on Lucknow-Sultanpur Road to some typical stones

Sl. No.	Details of compartment	Adhesion test: Tar No. 3 — Adhesion No.	Tar No. 3 — Adhesion	Mexphalt 80/100 — Adhesion No.	Mexphalt 80/100 — Adhesion	Remarks
1.	Compartment No. 2	1	Poor	0–1	Poor	
2.	" 3	1	Poor	0–1	Poor	
3.	" 4	2	Sufficient	0	Poor	
4.	" 5	1	Poor	1	Poor	
5.	" 6	1	Poor	1	Poor	
6.	" 7	0–1	Poor	1	Poor	
7.	" 8	1	Poor	5–6	Good	
8.	" 9	0–1	Poor	5–6	Good	
9.	" 10	0–1	Poor	4	Ordinary	
10.	" 11	0–1	Poor	0–1	Poor	
11.	" 12	1	Poor	2–3	Ordinary	Red & black mixed
12.	" 13	0	Poor	3	Ordinary	Dark red
13.	" 14	0	V. Poor	1	Poor	Pale brown & red mixed
14.	" 15	0	V. Poor	1–2	Sufficient	
15.	" 16	0	V. Poor	1	Poor	
16.	" 17	0	V. Poor	0–1	Poo	
17.	" 18	0	Poor	1–2	Suff cien	
18.	" 19	0–1	Poor	0	Poo	
19.	" 20	0	V. Poor	1–2	Suff cien	

TABLE 7—*contd.*

Sl. No.	Details of compartment	Adhesion test				Remarks
		Tar No. 3		Mexphalt 80/100		
		Adhesion No.	Adhesion	Adhesion No.	Adhesion	
20.	Bricks burnt ½ hrs. extra	0	V. Poor	1–2	Sufficient	Dark red with black spots
21.	,, 1 ,,	0	Poor	4	Ordinary	
22.	,, 1½ ,,	1	Poor	1–2	Sufficient	Blackish spotted
23.	,, 3 ,,	0	V. Poor	1	Poor	Dark red & black mixed.
24.	,, 6 ,,	0	V. Poor	3-4	Ordinary	Red & dark red mixed
25.	,, 9 ,,	0	V. Poor	2-3	Sufficient	Red with black spots
26.	,, 12 ,,	0	V. Poor	1–2	Sufficient	Dark red and red mixed.
27.	Blackish brick ballast	0	V. Poor	5	Good	
28.	,, ,, ,,	0	V. Poor	1	Poor	
29.	Shankargarh (Allahabad)	0	Poor	5	Good	
30.	Jharpahar (Jhansi)	1	Poor	5	Good	
31.	Bayana (Agra)	2–3	Sufficient	5	Good	
32.	Bhartkup (Banda)	0	V. Poor	1	Poor	
33.	Jhansi	5	Good	1	Poor	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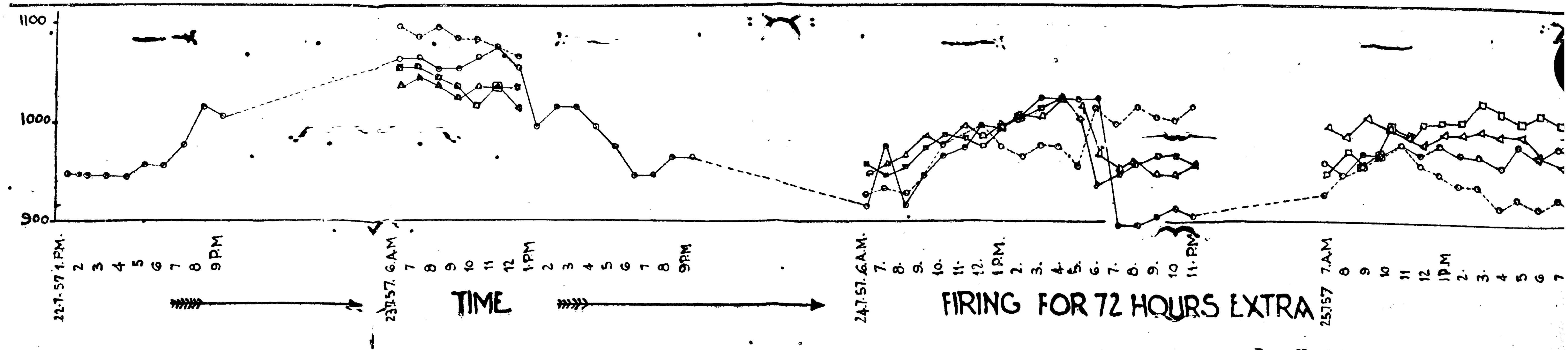
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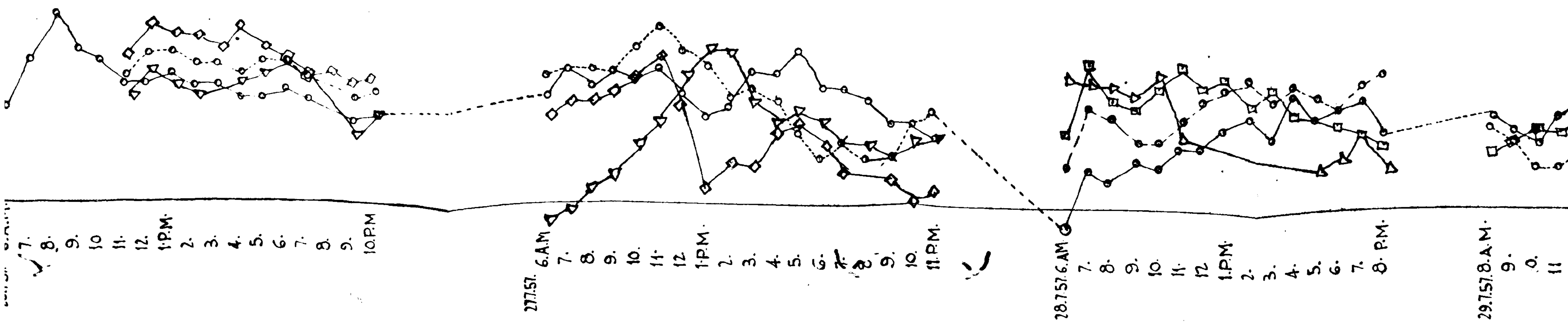
TIME
FIRING FOR 72 HOURS EXTRA

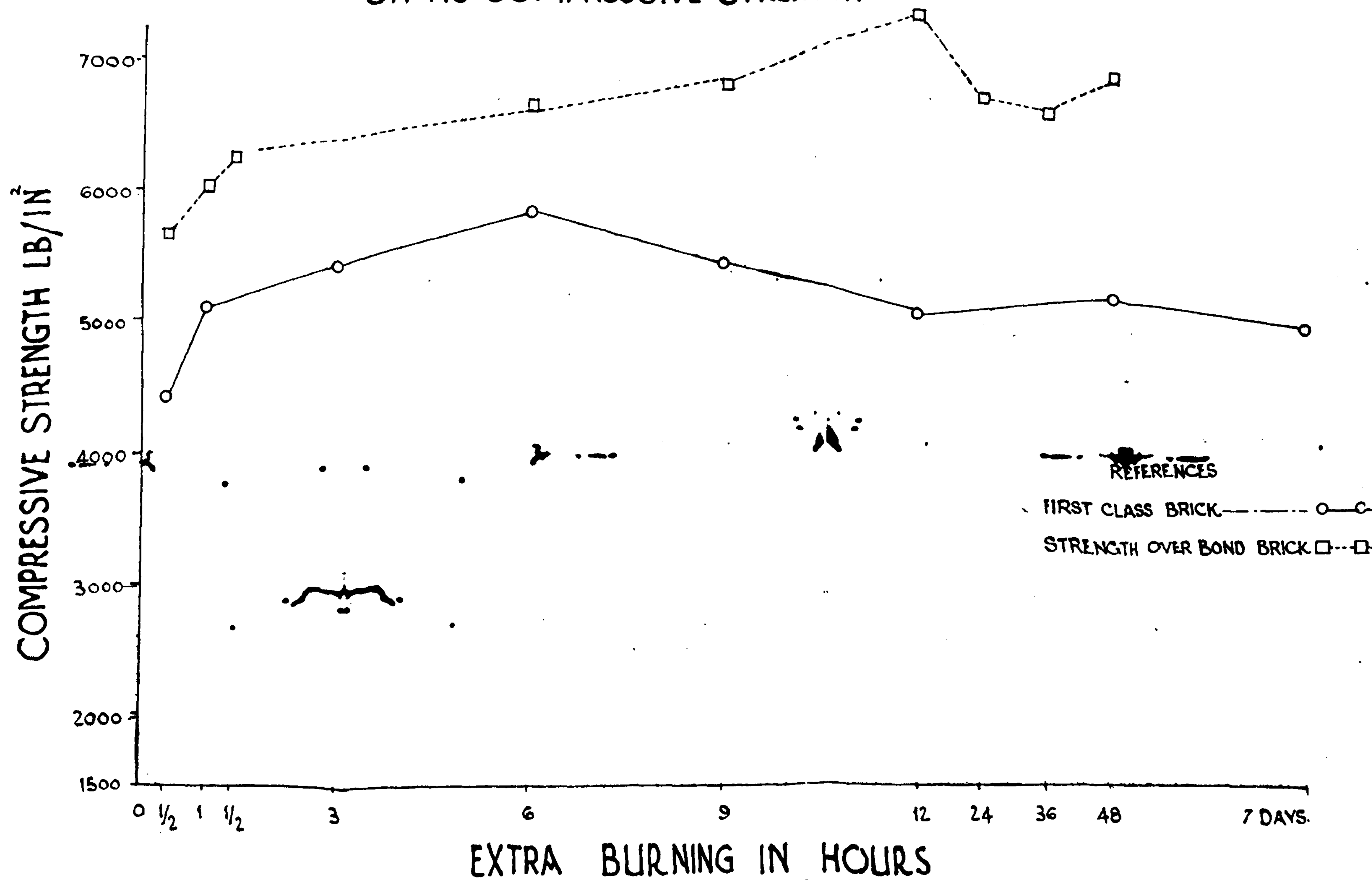
PLATE I



B)

CHARTS INDICATING TEMPERATURES INSIDE THE KILN
DURING EXTRA FIRING

EFFECT OF EXTRA-BURNING OF BRICKS ON ITS COMPRESSIVE STRENGTH



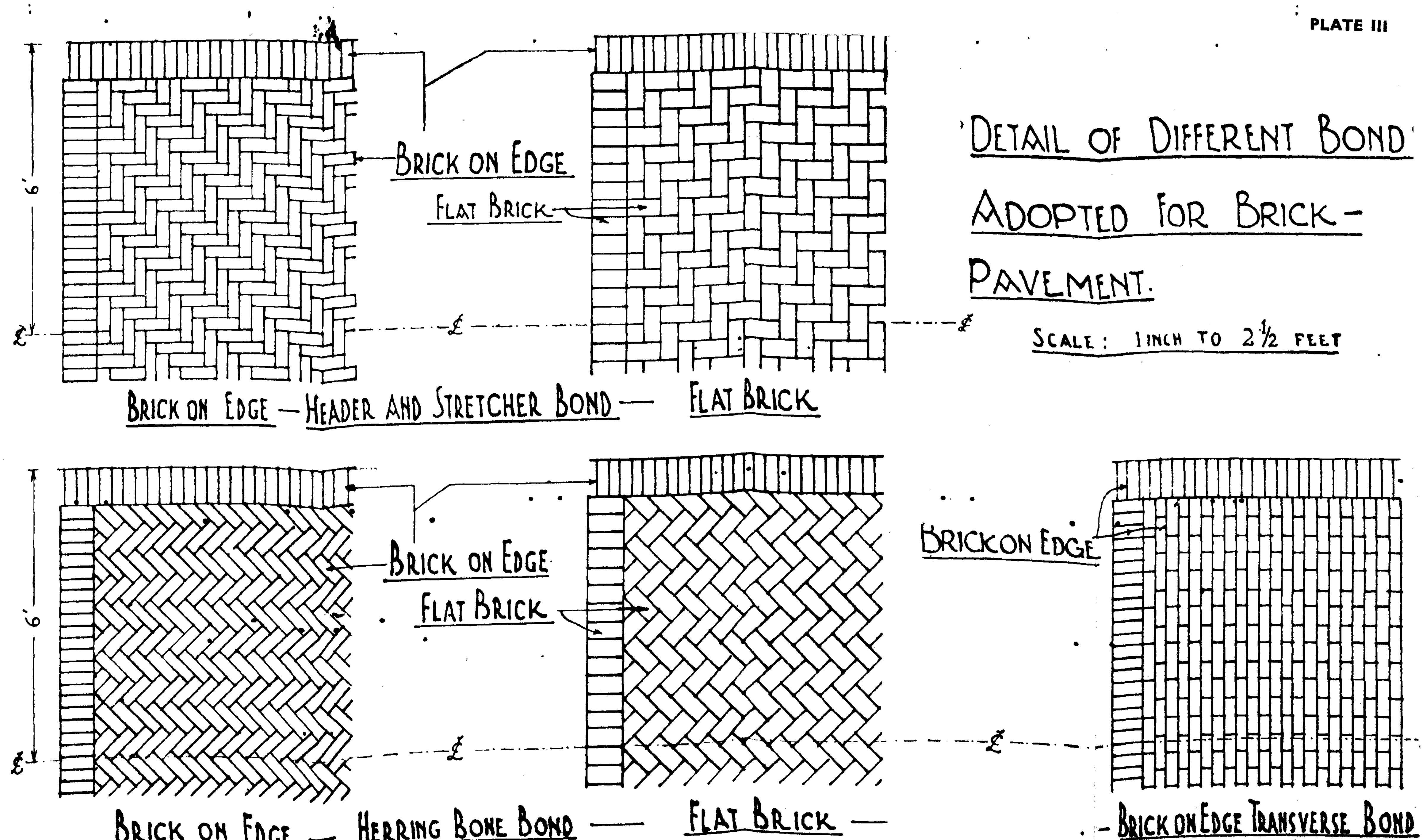
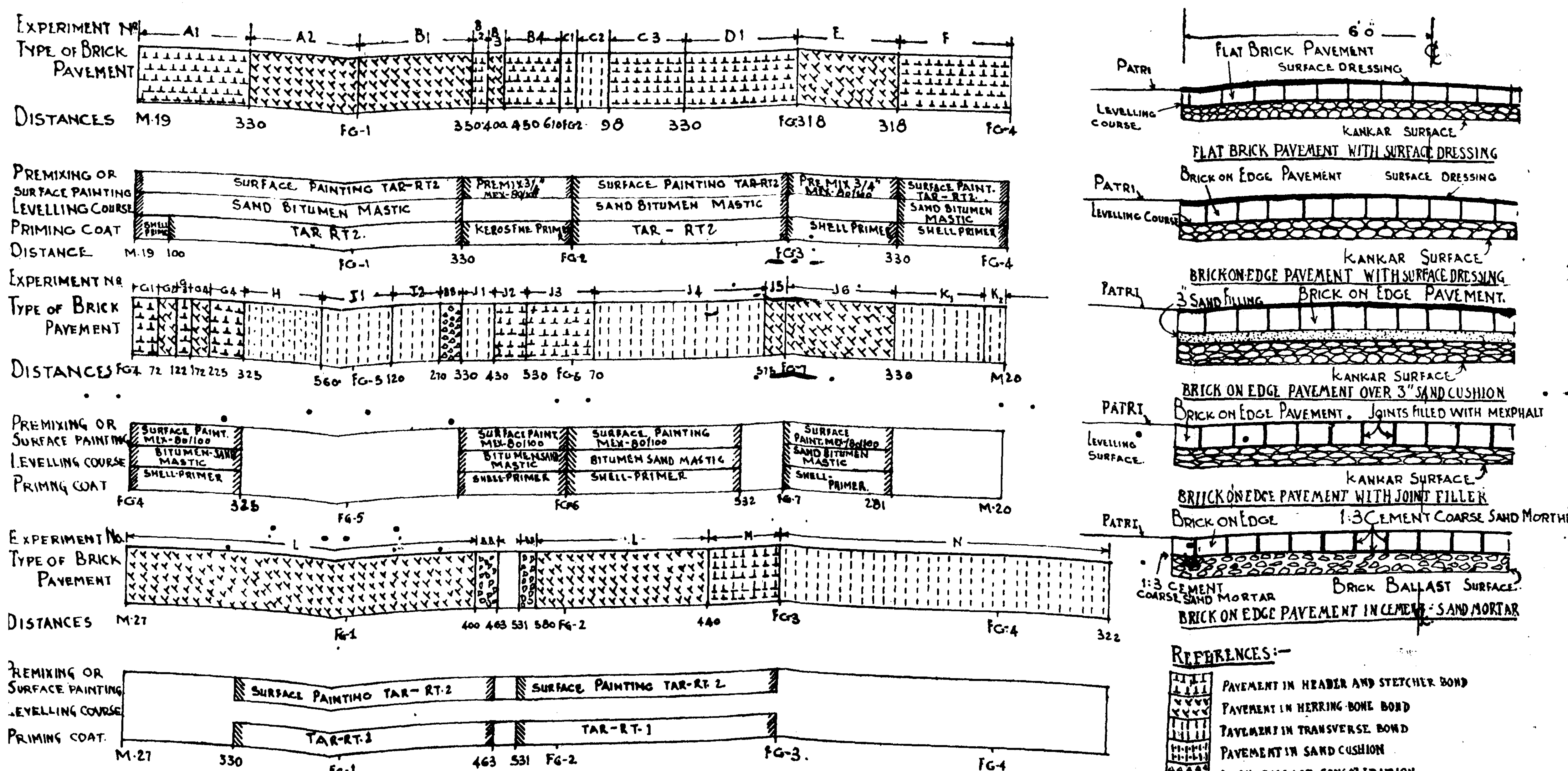

DETAIL OF DIFFERENT BOND ADOPTED FOR BRICK-PAVEMENT.
SCALE: 1 INCH TO 2½ FEET
BRICK ON EDGE
FLAT BRICK
6'
BRICK ON EDGE — HEADER AND STRETCHER BOND — FLAT BRICK
BRICK ON EDGE
FLAT BRICK
BRICK ON EDGE
6'
BRICK ON EDGE — HERRING BONE BOND — FLAT BRICK —
— BRICK ON EDGE TRANSVERSE BOND

PLATE IV

DETAILS OF EXPERIMENTAL LENGTHS OF BRICK PAVEMENT MILES 20 AND 28 OF LUCKNOW-SULTANPUR ROAD.



Paper No. 211

"INVESTIGATION OF FAILURES OF A BITUMINOUS ROAD"

By

C. G. SWAMINATHAN,* and S. ARUMUGA SWAMI *

CONTENTS

* Central Road Research Institute, New Delhi.

SYNOPSIS

This Paper deals with a detailed investigation that was carried out on an important road to study the causes of the extensive surface cracks that had developed on it. Trenches were cut in the road to expose the subgrade in various locations along the road, where the road was sound and at others where the road had failed. Data regarding the type and the thickness of the courses, the properties of the road material, the consistency of the extracted binder as well as field C.B.R. and plate bearing values of the subgrade soils saturated in places, were determined.

McLeod's chart for the design of flexible pavements was used to determine as to whether the failures of the pavement were due to inadequate thickness or not, based on the field C.B.R., plate bearing and cone-bearing values. An approximate correlation between the field C.B.R. and non-repetitive plate-bearing test values within specific limits is set forth based on the investigation data.

Causes of the failures were grouped into three categories, viz., those arising from an unsatisfactory foundation or inadequate thickness of pavement, those due to failures of the base or wearing aided by the weathering of the binder and those due to waterlogging.

In certain sandy subgrade sites the presence of hair cracks was observed during the investigation and an attempt is made to explain this.

1. INTRODUCTION

One of the important strategic roads in North India had developed a series of failures in the wearing surface and it was reflected in fairly wide cracks even on such of those patchworks as were laid as a remedial measure. The road was originally constructed for a width of about 12 ft with kankar base course of thickness varying from 9 to 18 in. on the local subgrade which was, in general, a silty soil. A wearing course of 1 to 3 in. thick of stone metal was laid on the kankar base.

The road had been surface dressed with a bituminous binder to suit the present day traffic needs. However, the intensity of traffic was so high that widening became a necessity and the road was subsequently widened in 1948 by 5 ft on either side or by 10 ft on one side depending on the needs of alignment of the road. Fig. 1 shows a typical cross-section of the road. The widening was effected by flat brick soling 3 in. thick with a wearing course of 3 to $3\frac{1}{2}$ in. of stone metal. Both the original and widened portions of the road were surface dressed with bitumen in 1949 and the road was surface dressed subsequently in 1952 with tar. Since then, except for occasional patchworks, no renewal coat was given till the time of this investigation.

The intensity of the traffic on this road was about 3000 tons per day, Table 15, which comprised mostly heavily loaded trucks,

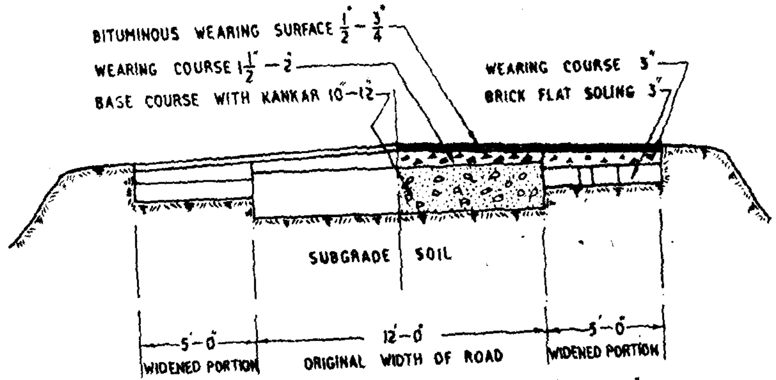


Fig. 1. A typical cross-section of the road

and passenger buses and military vehicles together with a small number of bullock carts.

Also, it was noticed that though the whole surface was cracked over a good number of miles, the camber and the gradient of the road in many places were unaffected by the cracks, at least at present, and the fast moving vehicles were running at a speed of over thirty miles an hour with comparative ease.

2. NATURE OF THE FIELD INVESTIGATION

2.1. Choice of Tests

In the past very few bituminous roads were built based on scientific principles of designs and even then they may not always cater to the demands of the present day traffic conditions. So, the reason for the failure of a bituminous surface could be attributed to one or more causes such as weak subgrade, inadequate thickness of pavement, lack of control in the construction techniques, etc.

Thus, the problem of surface cracks on bituminous surfacings is a multi-faced one, demanding a thorough analysis of the subgrade from the point of view of settlement, an analysis of the materials forming the base and wearing courses and also an analysis of the wearing surface itself. Therefore, in selected locations of the road, the following field and laboratory tests were carried out on the subgrade :

1. Field C.B.R.
2. Plate bearing—12 in. diameter plate.

3. North Dakota cone bearing.
4. Proctor needle penetration test.
5. Field density.
6. Moisture content.
7. Atterberg limits.
8. Sulphate content.
9. Sand content.

Tests 1 to 6 were conducted at the existing moisture content of the subgrade soil—the existing moisture content being that which existed in the road bed at the time of testing. In order to assess the strength of the subgrade under adverse moisture conditions, tests 1 to 6 were also done after completely inundating the selected location for three days.

On the base course material : 1. Aggregate impact Test.

On the wearing course materials : 1. Aggregate impact Test.

On the wearing surface materials : 1. Extraction of binder.
2. Equi-viscous temperature of the extracted binder.
3. Softening point of the extracted binder.

2.2. Test Locations

In the stretch of about 40 miles between M25—M65, twelve different test sites were selected at suitable intervals, Fig. 2.

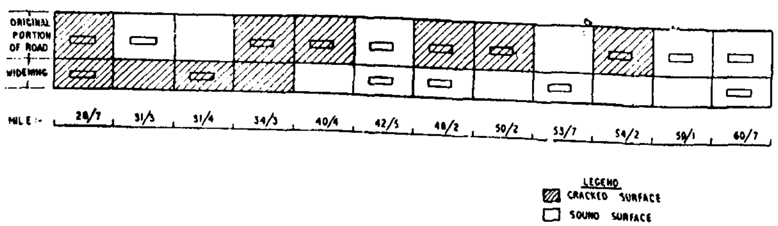


Fig. 2. **Location of test sites**

The details of the test locations are given in Table 1. Some of the test sites were located in sound sections of the road where the

TABLE 1

Details of the test locations

Test site No.	Mile/ Furlong	No. of pits in original	No. of pits in widening	Condition of pavement surface	Description
1	28/7	1	1	cracked	The entire surface was cracked in an alligator or map pattern. Settlement of the road bed was noticeable.
2	31/3	1	—	sound	—
3	31/4	—	1	cracked	Here the original portion of the road was in sound condition but the widening was awfully marred with cracks which were in an advanced stage. The cracks were ½-¾ in. wide and quite deep.
4	34/3	1	—	cracked	The cracks were severe and of map pattern on the whole width of the road.
5	40/4	1	—	cracked	The surface was full of map pattern cracks. The cracks were thin. There was no visible settlement or waviness on the road surface.
6	42/5	1	1	sound	—
7	48/2	1	1	Orig. cracked, Widg. sound	Here the original portion of the road was cracked but the widening was sound with no cracks. The cracks were extremely thin and no visible settlement was observed.
8	50/2	1	—	cracked	The site was in a waterlogged area. Heavy settlement of the road had occurred in this area. The cracks were severe, pittings were noticed.
9	53/7	—	1	sound	—
10	54/2	1	—	cracked	The cracks were of map pattern on the whole width of the road. Pittings were observed on the bituminous surface. But there was no visible settlement of the road bed.
11	59/1	1	—	sound	—
12	60/7	1	1	sound	—

road surface was in excellent condition in order to have a comparative study of the affected and unaffected stretches, whereas the rest were located in portions of the road where cracks had developed on the bituminous surfacing in varying degrees.

Since the road bed had distinctly two different base courses, namely, kankar base in the centre and brick-on-flat base in the widened portion, a sufficient number of test sites were located both in the original and the widened portions of the road.

2.3. Field Testing Techniques

In order to conduct tests on the soil of the subgrade in situ, trenches were cut longitudinally and right through the depths of the road construction to expose the subgrade at these sites. The trenches were confined to one half of the carriageway in order that the road might be kept open to traffic. The length of the trenches varied from 15 to 28 feet. Most of the trenches were dug to a length of 22 feet. In all cases the width was 3 feet and the depth taken was upto the subgrade, Fig. 3. Half the length of the trench was used for the various tests at the existing moisture content and the other half was utilised for the various tests after saturation. Fig. 4 shows the placement of the various tests in a test pit.

That half of the trench which was intended for tests after saturation was ponded with water for a minimum period of 72 hours to ensure complete saturation. The trench was dewatered and the top 1 in. of the soaked subgrade was removed to get rid of the slurry at the time of testing.

The various tests, viz., the field C.B.R., plate bearing, Proctor's needle, and North Dakota cone were conducted as per standard practice.[1] The load for the field tests was applied by means of a four-ton mechanical jack and measured with a five-ton proving ring, the necessary reaction being obtained by means of a five-ton loaded truck. The diameter of the plate used was 12 inches. 1/10 and $\frac{1}{4}$ sq. in. Proctor needles were used.

Moisture content and field density tests as well as E.V.T. and softening point of the extracted binder—soxhlet extraction being done using Pyridine—were also conducted in accordance with the standard practice.

To determine the quality of the stone, kankar and bricks that were used, the stretch of the road under investigation was divided

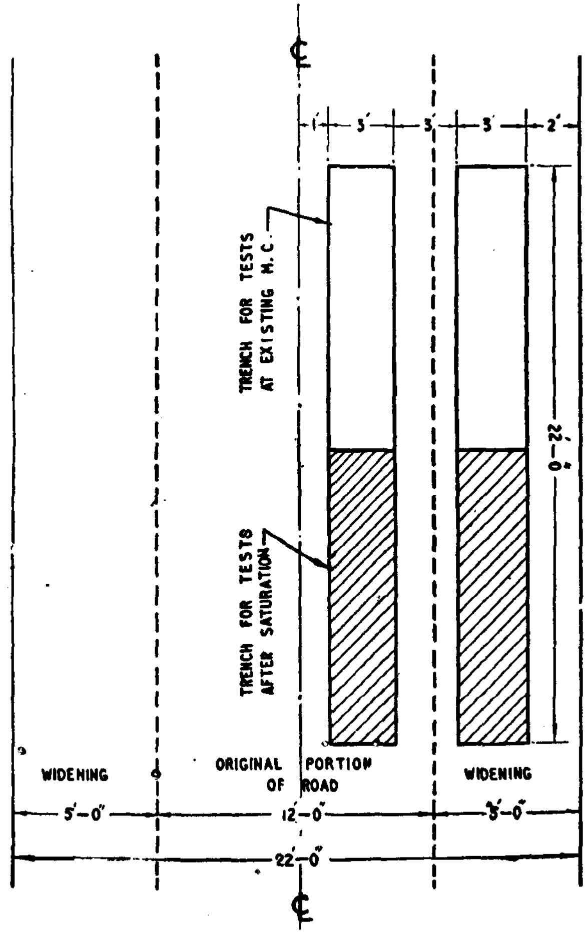


Fig. 3. Details of the test pits

into three zones, viz., M29–M35, M36–M48, and M49–M6 respectively. The representative samples were subjected to aggregate impact tests.

Since the variation in the moisture content of the subgrade soil plays a vital role in the performance of the road bed, data concerning the existing as well as field saturated moisture contents together with the depth of water table at the site were collected. Data pertaining to the seasonal variation in the sub-

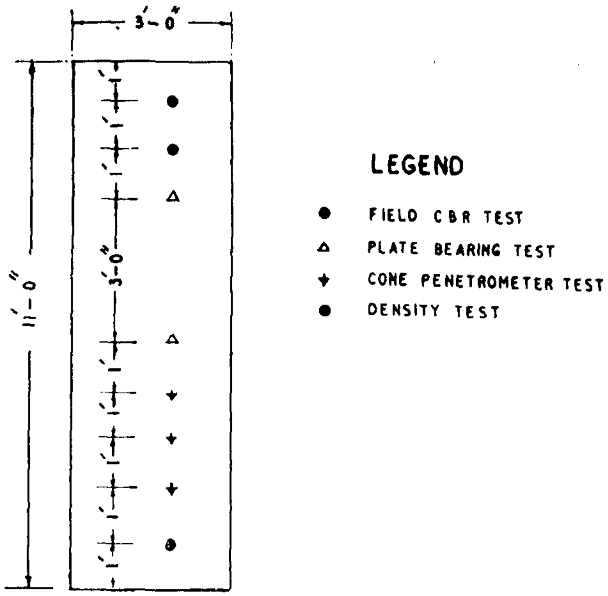


Fig. 4. **Plan of the disposition of various tests in a test pit**

grade moisture would have been more useful indeed but then such a data should be collected over a period of years.

3. DISCUSSION OF TEST RESULTS

3.1. General

3.1.1. Condition of Road Surface

Before entering into the discussion of the test data, it is essential to recall the general condition of this stretch of the road under investigation. It was noticed that the widened portion upto mile 39 was distinctly different in condition from the widening beyond mile 39. As for the original portion of the road, one could observe a good deal of settlement of the road and heavy patchworks in many places, being very bad between miles 32 to 35 and again between miles 36 to 37/4. In these stretches, in some places, both the original and the widened portions were affected.

Beyond mile 39, in general, the widening was in good condition without much visible settlements and in most places the road was sound. However, the stretch of the road between miles

50 and 52 was very badly affected owing to excessive settlements with deep corrugations of the road crust and heavy patchworks were seen in this region. Beyond mile 53, in general, the road was in a good condition.

However, it was observed at the time of this investigation that cracks had developed on the wearing surface in many places on the road and in some places the cracks were in a fairly advanced stage. The cracks were mostly of an 'alligator pattern' over the entire width of the road with longitudinal and diagonal cracks here and there. In many places, though the whole surface of the road between berms was marred with map pattern cracks, the width of the cracks varied. In some places, the width was as thin as about 1/10 in. whereas in other places the cracks were about ½ in. wide. Whereas in quite a few places the depth of the cracks was only about ½ in. (the cracks being confined to the wearing surface only), in some other places it was observed that the cracks had penetrated 2 to 2½ in. into the wearing course. With the traffic increasing every day, it was feared that its impact on the road will be not only to widen the cracks but also to develop new cracks, ultimately leading to the failure of the entire road.

3.1.2. Comparison of Test Sections

At the outset the various test sections are grouped into two categories, namely, (a) those in the original portion of the road having a kankar base course, and (b) those in the widened portion having a base course of brick-flats, for convenience of comparison of the test sections inasmuch as their structures are different from one another. Again, the intensity of traffic is not the same in both the portions at any point on the road, since heavy lorries and other fast moving vehicles always have a tendency to use the centre of the road yet, it is noticed that cracks had developed in both the groups.

The failure of the road structure can be due to one or more reasons since several factors of varying degrees of significance may combine to cause a road to fail or develop defects and so it is difficult to classify the investigations under clear cut headings. However, in order to simplify the discussions, the investigations are considered under three main headings, namely,

1. Failures due to an inadequate thickness of the road structure,
2. Failures due to subsidence within the pavement, together with weathering of the binder, and
3. Failures due to waterlogging.

3.2. Pavement Design Methods

In the absence of precise methods for the evaluation of the exact stresses induced in the pavement with the consequent difficulty in the correct assessment of the strength of the structure involved, pavement design methods have been mostly empirical or partly coupled with theoretical considerations. The four or five methods that were adopted during this investigation are being studied critically in the light of the data as obtained from this investigation only.

3.2.1. C.B.R. Method

The results of the laboratory C.B.R. tests are used normally in conjunction with empirical design curves for estimating the required thickness of the flexible pavements for a given subgrade soil for a given traffic condition, Fig. 5. These C.B.R. curves are based on C.B.R values of compacted and saturated specimens of

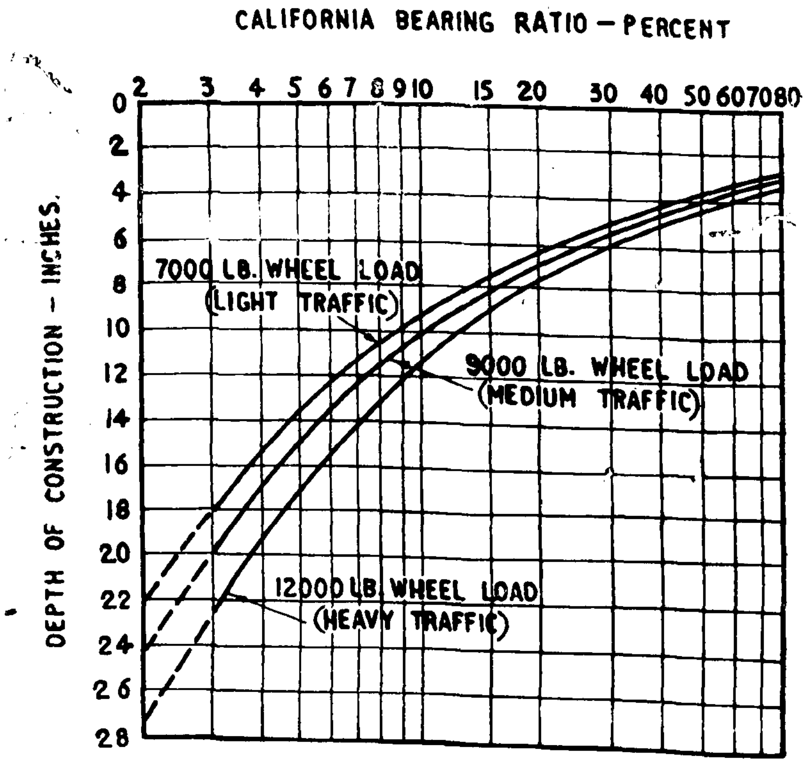


Fig. 5. Design curves for flexible pavements. The California Bearing Ratio method

the subgrade soil. It is wellknown that the C.B.R. test is open to quite a number of valid criticisms.[2] Further, where field C.B.R. values are available, it is difficult to interpret the data since research work as regards the relationship between the laboratory and field C.B.R. values has been limited and not very enlightening. Table 2 clearly indicates the wide variation between laboratory and field C.B.R. values for any soil type.

However, design curves connecting field C.B.R. and the thickness of the flexible pavement have been evolved of late by McLeod[3] for various wheel loads, Fig. 6. A comparison of the two charts, viz., Fig. 5 and Fig. 6, indicates that the required thickness of the

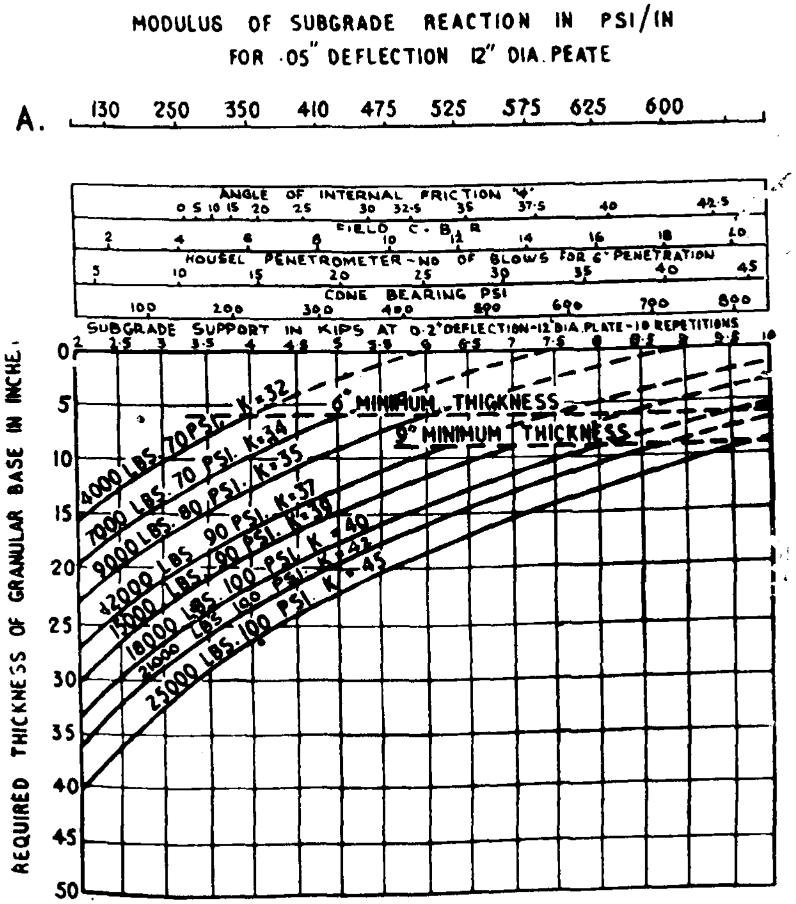


Fig. 6. Design curves for flexible pavements for highway wheel loadings for highest traffic capacity (single wheel)

pavement based on field C.B.R. value is less conservative than that based on laboratory C.B.R. value. For this reason, the former is made use of in this investigation.

TABLE 2

Laboratory and field C.B.R. values of the subgrade soil

No.	Mile/ Furlong	Condition of pavement	C. B. R. VALUES			
			Field		Laboratory	
			At existing M.C.	After saturation	Before saturation	After saturation
1	28/7	Widening-cracked	9.2	2.0	8.0	0.85
2	40/4	Original-cracked	55.9	38.0	3.2	0.93
3	50/2	Original-cracked	38.5	—	11.2	1.8
4	31/3	Original-sound	52.5	11.2	5.4	0.55
5	42/5	Widening-sound	84.0	13.0	100.0	11.6
6	60/7	Original-sound	31.3	4.5	87.5	1.6

3.2.2. Plate Bearing Method

The most important point in favour of the plate bearing test as against the C.B.R. test is that, in the former, the relatively larger contact area between the plate and the subgrade soil brings the soil under test nearer to the reality in so far as its behaviour under an applied load is concerned.

The plate bearing test is usually conducted on plates of 30 in. diameter for the design of airfields[4] and on 18 in. diameter plates for highways.[5] Since plates of large diameters require high dead load reactions, smaller sized plates (such as 12 in. diameter plates) can be very conveniently employed and the values obtained can be corrected to obtain the value corresponding to a plate of any diameter upto 30 in. based on Stratton's empirical relationship.[6] In this investigation a 12 in. diameter plate was used and, wherever necessary, the values are corrected accordingly.

In order to utilise the data fully from the point of view of pavement thicknesses, the 'K' values were translated in terms of the laboratory C.B.R. values, as given in line X of Fig. 7.

The curves connecting laboratory C.B.R. and 'K' values are given in Fig. 8.

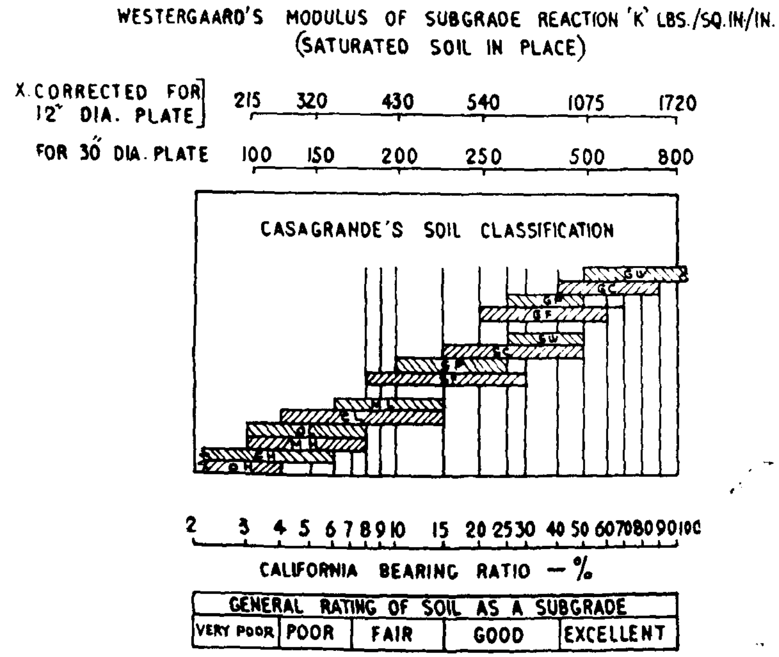


Fig. 7. **Approximate correlation between Casagrande's airfield soil classification, C. B. R., and Westergaard's modulus of subgrade reaction**

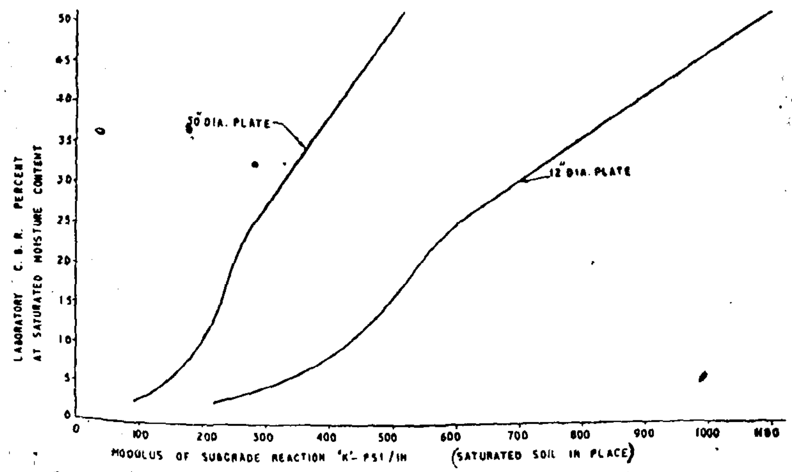


FIG. 8. RELATIONSHIP BETWEEN LABORATORY C B R AND 'K' VALUES

8

It will be seen from Fig. 6 that in addition to the field C.B.R. data, the 'K' values as obtained from the plate bearing test for repetitive loading are also correlated with the thickness of pavement, with the result that using the 'K' values of the repetitive plate bearing test, the thickness required can be got directly from the chart. However, in view of the fact that in this investigation the plate bearing test was done for non-repetitive type of loading, an attempt is again made here to correlate the field C.B.R. values with the static plate bearing values. In Fig. 9, which shows the relationship between the values of field C.B.R. and plate bearing,

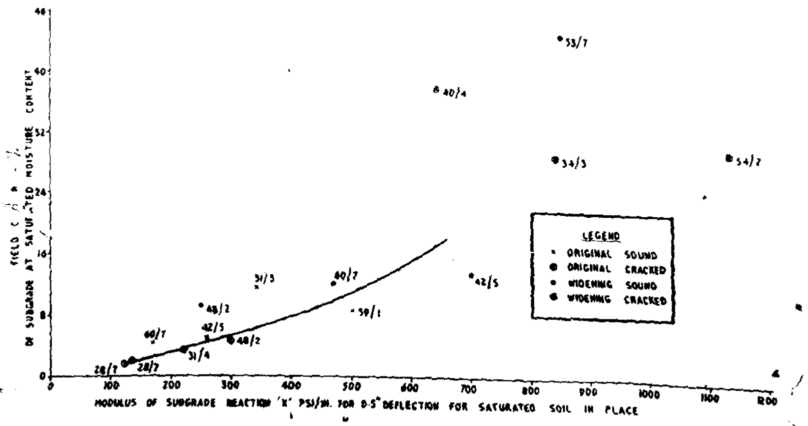


FIG-9. RELATIONSHIP BETWEEN FIELD C. B. R. AND 'K' VALUES

as obtained in this investigation, it can be seen that the field C.B.R. and the 'K' values follow more or less a definite path within specific limitations. Assuming that the relationship holds good, it is possible to interpret the 'K' values as obtained in a non-repetitive plate bearing test by translating it in terms of the field C.B.R. values to stipulate the actual thicknesses required. The approximate correlation between them, as far as the results of this investigation are concerned, is indicated in line A at the top of Fig. 6. From that figure it is also possible to obtain an approximate correlation between the static plate bearing and the repetitive plate bearing test values, as given in Fig. 10.

In Fig. 11, the curves for the thicknesses as would be required for a wheel load of 9000 lb for the two sets of 'K' values discussed above, namely, those corresponding to laboratory C.B.R. and field C.B.R. respectively, are given. Here, again, it will be

noted that the thickness required for a given 'K' value, for a given wheel load, would be less when the thickness is seen from McLeod's chart instead of the conventional chart.

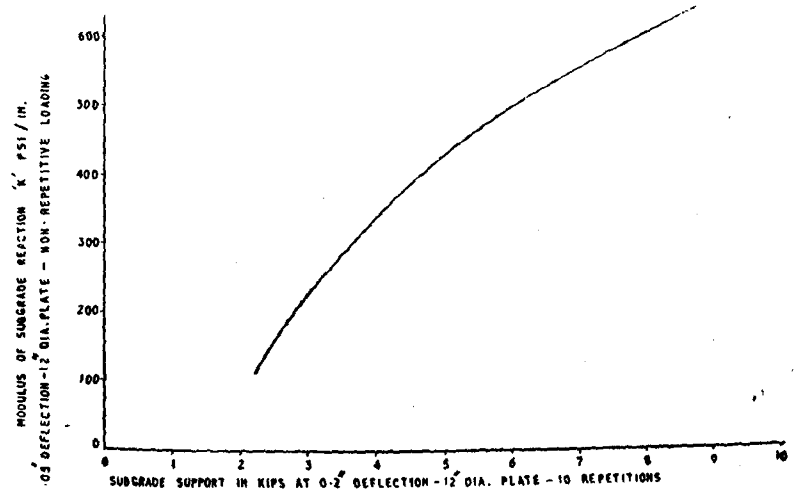


FIG.10. RELATION BETWEEN 'K' VALUES OF REPETITIVE AND NON REPETITIVE PLATE BEARING TESTS

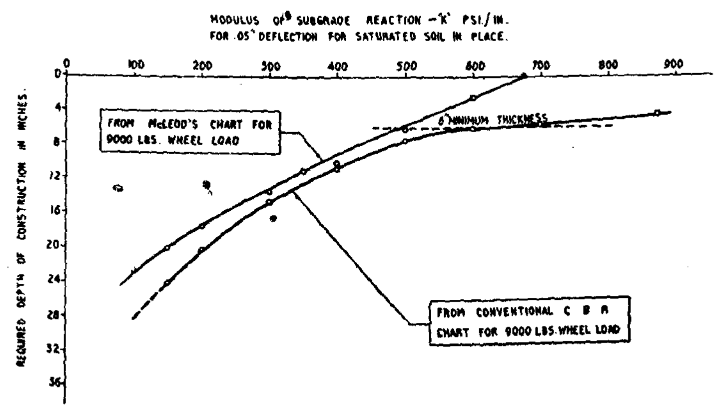


FIG:-11. PAVEMENT THICKNESS CURVES FOR 9000 LB WHEEL LOAD IN TERMS OF 'K' VALUES

3.2.3. North Dakota Cone Method

The cone bearing value is related to the thickness of pavement by an equation

$$T = \frac{65.7}{B^{0.388}}$$

where T equals the thickness of construction in inches and B equals the cone bearing values in lb per sq. in. The thickness can also be obtained directly from the curves as shown in Fig. 6.

3.2.4. Proctor's Penetration Test

Proctor's penetration method was employed in this investigation to see if it is possible to correlate the necessary data with field performance but, from the data collected, no definite correlation seems to exist, and it may not always be possible to fix the reasons for the failures exactly based on the Proctor's penetration value.

3.3. Failures due to Inadequate Thickness of Pavement

3.3.1. Test Sections in the Original Portion of the Road

A perusal of the thickness of the road beds in the various test sections in the original portion of the road shows that those in miles 28/7, 31/3, 40/4, 50/2 and 54/2 are about 11½ in. thick, those in miles 59/1, 42/5, and 48/2 are about 14 in. thick and that in mile 60/7 is 21 in. thick, Table 3. Both in the 11½ in. and 14 in. thickness groups some sections have developed cracks while some have not, and M60/7 is sound. It may be remembered here that all the test sections are situated on the same road, having the same intensity of traffic and subjected to identical conditions of climate and other weather effects, except M 50/2 which is situated in a waterlogged area, for which reason it is dealt with separately. Hence, to analyse the causes of the failure of the various test sections, it is essential to study their subgrade soil conditions first, which is directly related to the thickness of the pavement required for successful performance. For this purpose, either the values of C.B.R. or the modulus of subgrade reaction or the cone bearing value of the subgrade soil are useful.

Considering from the C.B.R. point of view, Table, 4 and 5, the curve pertaining to 9000 lb wheel load in Fig. 6 may be used for

TABLE 3

Thickness of the pavement in the original and widening of the road

No.	Location of test site	Condition of pavement	Original portion: Thickness of Kankar B.C. in.	Original portion: Thickness of Wearing course in.	Original portion: Thickness of Wearing coat in.	Original portion: Overall thickness of pavement in.	Widening: Thickness of Brick soling in.	Widening: Thickness of Wearing course in.	Widening: Thickness of Wearing coat in.	Widening: Overall thickness of pavement in.
1	Mile 28/7	cracked	9-10	1-1½	½-¾	12-14	3	3-3½	½-¾	6½-7½
2	31/3	sound	10-12	1½	¾-1	12-14	—	—	—	—
3	31/4	cracked	—	—	—	—	3	3-3½	½-¾	6½-7½
4	34/3	cracked	9-10	2	½-¾	11-12	—	—	—	—
5	40/4	cracked	9-10	1-1½	½-1	11-12	—	—	—	—
6	42/5	sound	12-13½	1½-2	½-¾	14-16	3	3-3½	½-¾	6½-7½
7	48/2	widg. sound & orig. cracked	12-14	3	½-¾	15-17	3	3-3½	½-¾	6½-7½
8	50/2	cracked	8-9	2-3	½-¾	11-12	—	—	—	—
9	53/7	sound	—	—	—	—	3	3-4	⅜-½	6½-7½
10	54/2	cracked	8-9	2½-3	½-1	11-13	—	—	—	—
11	59/1	sound	10-11	2½-3	1-1¼	13-15	—	—	—	—
12	60/7	sound	18	2-2½	1-1½	21-22	3	2-2¾	½-¾	6-7

TABLE 4

Results of strength tests on the subgrade of original portion of the road

Mile/Furlong	Condition of pavement	C.B.R. value (field) at		Plate bearing 'K' value for .50″ settlement at		Cone bearing value at		Proctor needle bearing value for 1 in. penetration at				Remarks
								1/10 sq. in.		1/4 sq. in.		
		Existing M.C. per cent	After saturation per cent	Existing M.C. p.s.i./in.	After saturation p.s.i./in.	Existing M.C. p.s.i.	After saturation p.s.i.	Existing M.C. p.s.i.	After saturation p.s.i.	Existing M.C. p.s.i.	After saturation p.s.i.	
28/7	cracked	21.6	1.8	499	120	314	75	520	66	372	40	
34/3	do.	42.5	29.0	1160	840	1008	112	900	200	CNP*	173	
40/4	do.	55.9	38.0	700	640	1300	502	1000	567	CNP	387	
48/2	do.	46.5	4.8	920	300	1714	344	CNP	400	CNP	280	
50/2	do.	38.5	—	300	—	1063	—	700	—	CNP	—	After saturation tests were not done
54/2	do.	42.6	25.0	825	1130	604	998	700	700	CNP	CNP	
31/3	sound	52.5	11.2	1540	340	1003	127	1150	300	CNP	300	
42/5	do.	50.5	4.8	1900	260	2030	270	CNP	212	CNP	220	
59/1	do.	58.7	8.5	1300	500	1040	541	766	183	CNP	166	
60/7	do.	31.3	4.5	1402	170	2121	407	CNP	250	CNP	213	

* CNP : could not penetrate.

TABLE 5

Results of strength tests on the subgrade of widening of the road

Mile/Furlong	Condition of pavement	C.B.R. value (field) at		Plate bearing 'K' value for .05″ settlement at		Cone bearing value at		Proctor needle bearing value for 1 in. penetration at				Remarks
								1/10 sq. in.		1/4 sq. in.		
		Existing M.C. per cent	After saturation per cent	Existing M.C. p.s.i./in.	After saturation p.s.i./in.	Existing M.C. p.s.i.	After saturation p.s.i.	Existing M.C. p.s.i.	After saturation p.s.i.	Existing M.C. p.s.i.	After saturation p.s.i.	
28/7	cracked	9.2	2.0	584	135	230	127	390	66	333	93	
31/4	do.	39.2	3.2	460	220	1242	failed	1133	73	CNP	69	
42/5	sound	84.0	13.2	1674	700	2750	191	CNP*	570	CNP	CNP	
48/2	do.	65.5	9.0	1080	250	2150	412	833	466	CNP	300	
53/7	do.	70.0	45.0	1100	850	2144	824	CNP	583	CNP	353	
60/7	do.	110.5	12.0	1500	470	5954	254	CNP	817	CNP	CNP	

* CNP : could not penetrate.

the purpose of interpretation of the field C.B.R. values for the present day traffic of 3000 tons per day on this road. Based on this curve, the thicknesses required for the various test sites are noted and given together with the existing thicknesses of the pavement in these sections in Table 6.

TABLE 6

Required thickness of pavement corresponding to the field C.B.R.

Site	Condition of pavement	Existing thickness in.	Required thickness from McLeod's chart in.
28/7	Cracked	11.5	20
34/3	,,	11.5	6
40/4	,,	11.5	6
48/2	,,	15	14
54/2	,,	12	6
31/3	Sound	13	6
42/5	,,	15	14
59/1	,,	14	9
60/7	,,	21	15

It is evident from the above table that except at M28/7, all the other sites have adequate thicknesses of pavement and yet failure has occurred in miles 34/3, 40/4, 48/2 and 54/2. This indicates that the failure in these sites may not be due to inadequate thickness of pavement but may be due to some other reasons.

Further, considering the plate bearing test values, it is stated that a weak subgrade has been found to have a 'K' value less than 125 p.s.i./in. (as corrected for a 12 in. diameter plate) whereas with a strong subgrade a value in excess of 625 p.s.i./in. can be obtained.[4] Applying the adequate correction for a 12 in. dia. plate based on Stratton's curves in Fig. 7, it may be stated that the subgrades having a 'K' value less than 370 p.s.i./in. for saturated soil in place should generally form poor and very poor

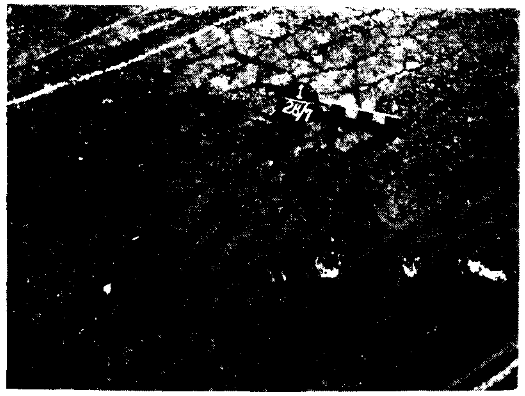


Photo 1.
Surface cracks in M 28/7, resulting from inadequate thickness of pavement

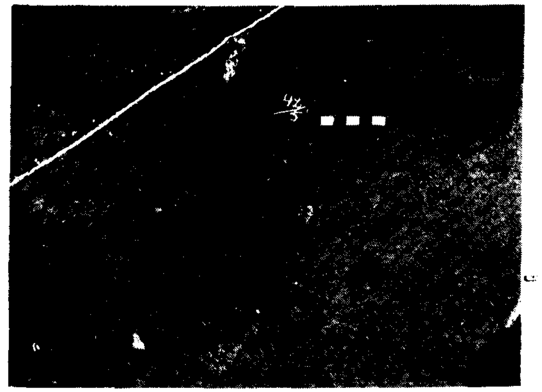


Photo 2.
Surfacing in sound condition at M 42/5

Photo 3.
Surface cracks in M 31/4, resulting from inadequate thickness of pavement

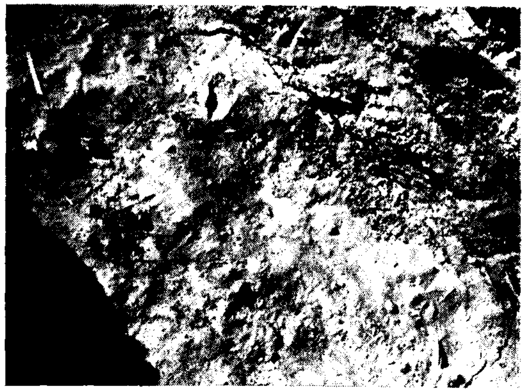


Photo 4.
Slump of subgrade soil at M 31/4 in the test pit. Note the cracks developed around the slumped soil

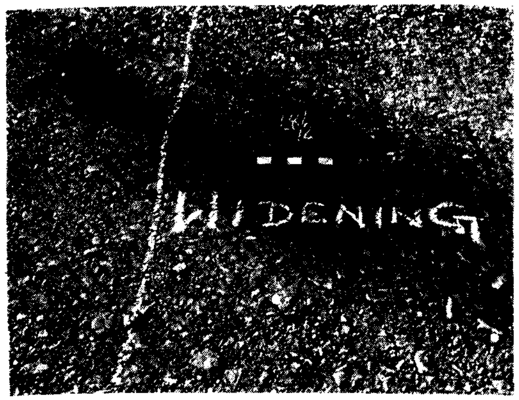


Photo 5.
Surfacing in sound condition at M 48/2

Photo 6.
Surface cracks in M 48/2 as a result of weathered binder

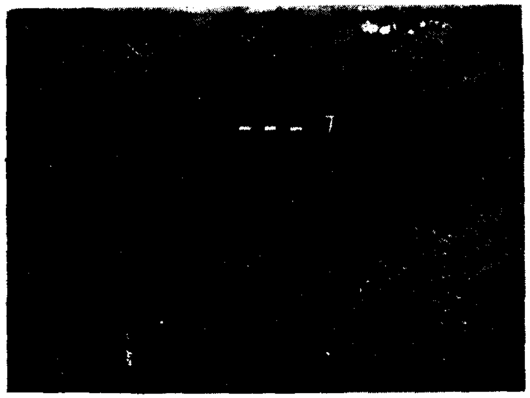


Photo 7.
First stage in the development of cracks on a sound pavement due to the subsidence of the base or wearing course

Photo 8.
Further development of cracks, emanating from the subsidence of the base or wearing course

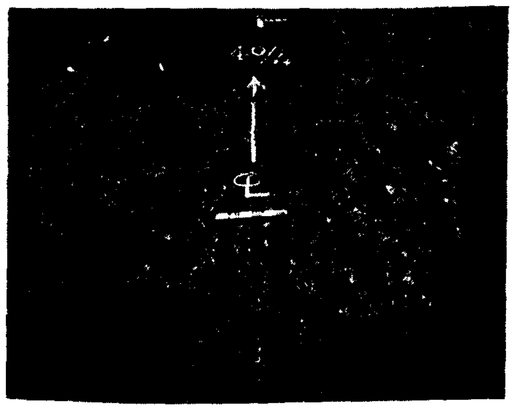


Photo 9.
Development of cracks on the whole width of the road

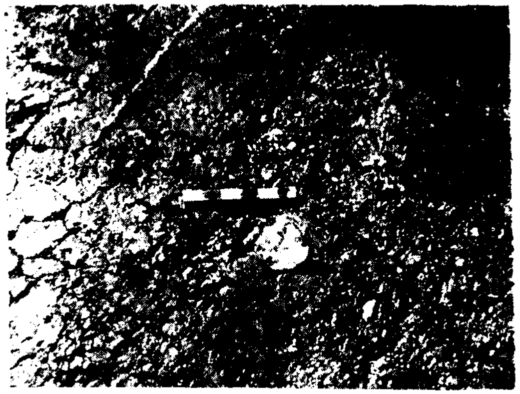


Photo 10.
Pitting of the surfacing leading to the end of its life

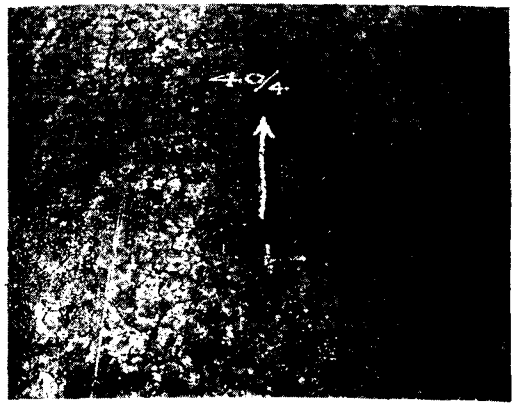


Photo 11.
Surface cracks in M 40/4, resulting from subsidence and failure within the pavement and weathering of the binder

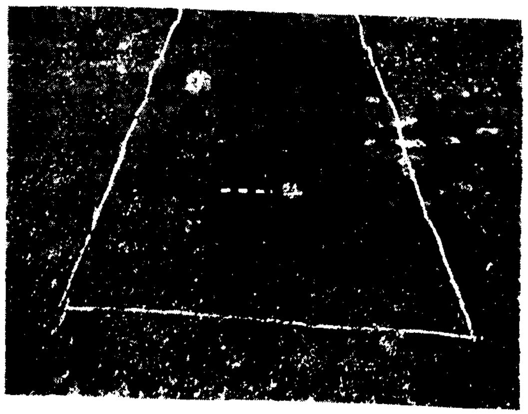


Photo 12.
Surface cracks in M 54/2, resulting from subsidence and failure within the pavement and weathering of the binder

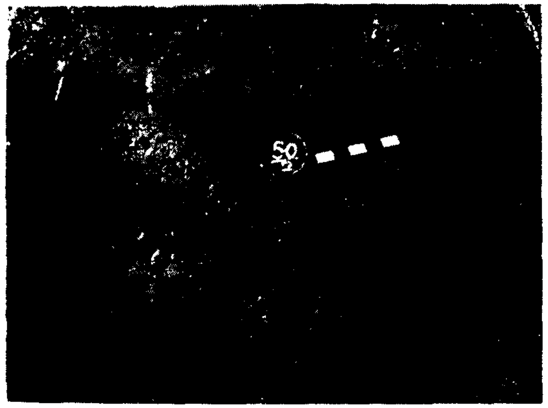


Photo 13.
Surface cracks in M 50/2, in a waterlogged area

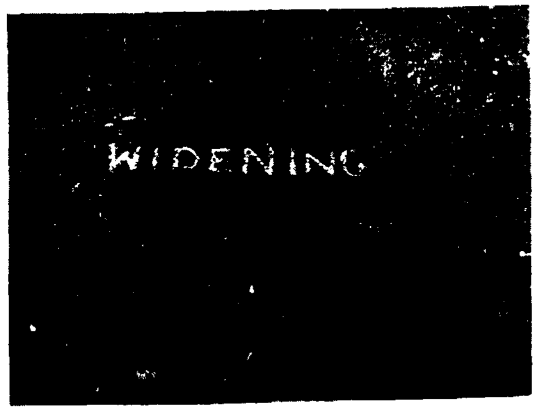


Photo 14.
Cracks on the patchwork in M 50/2

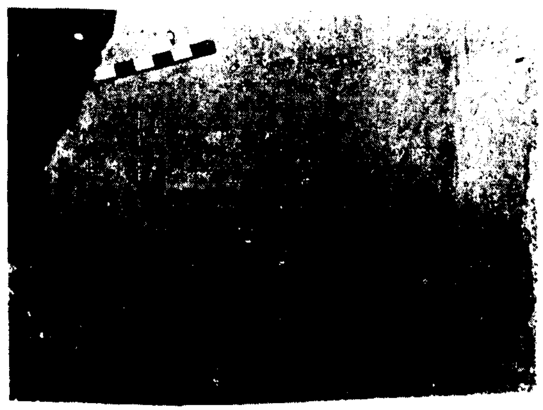


Photo 15.
Fine hair cracks on the subgrade soil in their natural state

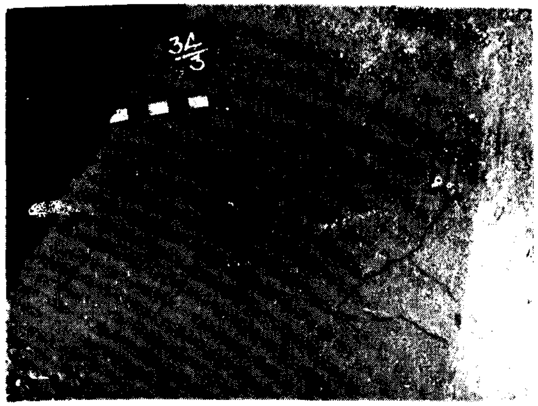


Photo 16.
The same hair cracks of Photo 15 marked with a pencil for clear presentation

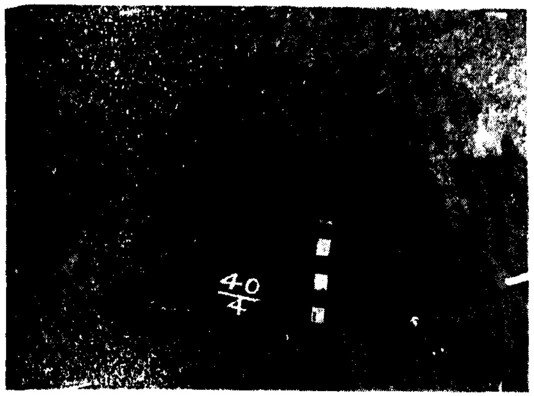


Photo 17.
Vertical section of the subgrade, depicting the extent of the depth of hair cracks

Photo 18.
The same section in Photo 17 depicting the hair cracks marked with a pencil

ubgrades, while those having a 'K' value above 625 p.s.i./in. hould be good and excellent subgrades. Though it may be said hat by providing for an adequate thickness of pavement, a good oad structure could be obtained even on a poor subgrade (as is he case of M 60/7), the above two limits set forth for the 'K' ralues are pertinent so far as this investigation is concerned, nasmuch as, in spite of identical traffic intensities and the same)verall thicknesses of the road structure, the road has failed in ertain stretches but is sound in certain other portions.

Moreover, all the subgrades having 'K' values for saturated oils in place below 370 p.s.i./in. fall distinctly under CL soils and hose having values above 625 p.s.i./in. group under SF and SW oils, Figs. 7 and 12. CL soils are normally rated to make poor ubgrades whereas SF and SW soils form good subgrades. This

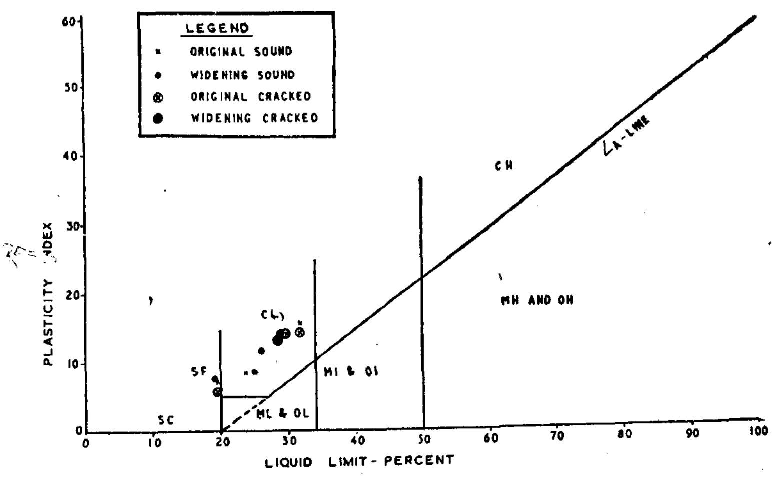


FIG. 12. PLASTICITY CHART USED IN CASAGRANDE SOIL CLASSIFICATION

is because CL soils being clayey in nature lose their strength when saturated. Their saturated moisture contents are higher than those of SF and SW soils as may be seen from Tables 7 and 8, which also shows that the saturated moisture contents of all the subgrade soils of the SF and SW groups fall below 18 per cent, whereas those of the CL soils are all above 18.0 per cent and those of the failed sections in the latter group are above 18 per cent.

TABLE 7

Results of dry density, moisture content, Atterberg limits, sand content and sulphate content tests on the subgrade of original portion of the road

Mile/ Furlong	Condition of pavement	Dry density at		Moisture content at		Atterberg limits			Depth of water-table ft	Sand content per cent	Sulphate content per cent	Remarks
		Existing M.C. gms./cc.	After saturation gms./cc.	Existing M.C. per cent	After saturation per cent	Liquid limit per cent	Plastic limit per cent	Plasticity index				
28/7	cracked	1.52	1.62	14.90	22.70	—	—	—	23	—	—	
34/3	,,	1.82	1.69	9.90	13.63	18.80	13.17	5.63	15	58.00	nil	
40/4	,,	1.61	1.85	4.44	13.60	—	Non plastic		15	69.72	nil	
48/2	,,	1.96	1.82	8.70	18.30	29.50	15.40	14.10	—	9.98	0.1	
50/2	,,	1.75	—	16.28	—	31.75	17.57	14.18	8	11.96	0.15	After saturation tests not done.
54/2	,,	1.84	1.95	6.38	11.70	—	Non plastic		10	66.74	nil	
31/3	sound	1.63	1.60	11.70	17.64	23.50	14.95	8.55	20	24.90	0.10	
42/5	,,	1.98	1.98	6.38	14.94	19.80	12.47	7.33	15	43.56	nil	
59/1	,,	1.79	1.85	3.60	17.64	—	Non plastic		16	68.82	nil	
60/7	,,	2.01	1.92	5.81	17.64	31.75	16.60	15.15	14	17.46	0.15	

TABLE 8

Results of dry density, moisture content, Atterberg limits, sand content and sulphate content tests on the subgrade of widening of the road

Mile/ Furlong	Condition of pavement	Dry density at		Moisture content at		Atterberg limits			Depth of water table	Sand content per cent ft.	Sulphate content per cent
		Existing M.C. gms./cc.	After saturation gms./cc.	Existing M.C. per cent	After saturation per cent	Liquid limit per cent	Plastic limit per cent	Plasticity index			
28/7	cracked	1.68	1.67	14.90	22.70	28.80	15.57	13.23	23	15.30	nil
31/4	do.	1.82	1.56	12.40	21.95	29.00	15.40	13.60	20	12.30	0.2
42/5	sound	2.05	1.98	5.26	15.65	19.80	12.02	7.78	15	40.62	0.3
48/2	do.	1.82	1.94	9.30	17.60	26.00	14.27	11.73	—	21.50	0.2
53/7	do.	2.05	2.05	6.38	12.36	Non plastic			11	66.74	nil
60/7	do.	1.89	1.83	4.71	18.30	24.97	16.40	8.57	14	34.76	0.2

By plotting the modulus of subgrade reaction of the subgrade soils against their saturated moisture contents, Fig. 13, it is clearly seen that all the test sites could be divided distinctly into four zones, such as :

Zone A ... Subgrades having 'K' value > 370 p.s.i./in. and saturated M.C. < 18 per cent

Zone B ... Subgrades having 'K' value > 370 p.s.i./in. and saturated M.C. > 18 per cent

Zone C ... Subgrades having 'K' value < 370 p.s.i./in. and saturated M.C. < 18 per cent

Zone D ... Subgrades having 'K' value < 370 p.s.i./in. and saturated M.C. > 18 per cent

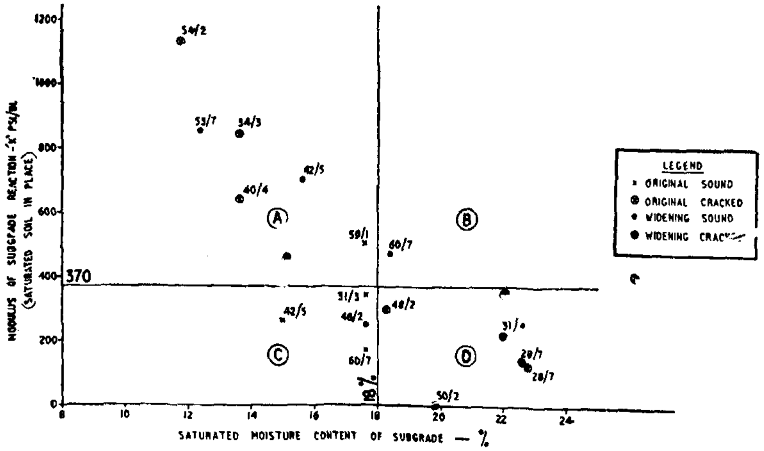


FIG-13 RELATIONSHIP BETWEEN 'K' VALUES AND SATURATION MOISTURE CONTENTS

It is immediately evident that all the sites falling in zone D have failed, and as seen earlier they all have subgrades of CL soils having poor subgrade values. The other three failed sections fall under zone A which, by virtue of the higher 'K' values and lower saturated moisture contents, should be the best zone. This confirms the possibility of the reasons for the failure of sites at miles 34/3, 40/4 and 54/2 as being different from those in the zone D.

Further, referring to Fig. 10, it may be seen that the thickness required for the various test sections can be obtained from the plate bearing values as given in Table 9.

TABLE 9

Required thickness of pavement corresponding to the modulus of subgrade reaction

Site	Condition of pavement	Existing thickness in.	Required thickness from McLeod's chart in.	Required thickness from conventional chart in.
28/7	Cracked	11.5	21	26
34/3	,,	11.5	6	6
40/4	,,	11.5	6	6
48/2	,,	15	14	15
54/2	,,	12	6	6
31/3	Sound	13	12	13
42/5	,,	15	14.5	17
59/1	,,	14	6.5	8
60/7	,,	21	18.5	22

From Table 9 it becomes apparent that the design based on McLeod's curves gives values lower than those based on the conventional curve. The test sites at miles 42/5 and 60/7 should have apparently failed, if based on the conventional curve, whereas they are intact. So, an analysis of the failures based on field data appears to be more reliable than that based on laboratory C.B.R. values which, as stated earlier, are more conservative. However, it becomes evident that only the pavement at M 28/7 is of inadequate thickness and all the rest are sound as far as the requirement of thickness is concerned. This adds further support to the earlier observation that the causes of failure of road at miles 34/3, 40/4, 48/2 and 54/2 are other than an inadequate thickness of the pavement.

Further, analysing from the point of view of the cone bearing values of the subgrade soil, it may be seen from Fig. 6 that the thickness required for the various test sections are as given in Table 10, which gives the required thickness both from the chart as well as calculation based on the equation given in Section 3.2.3.

TABLE 10

Required thickness of pavement corresponding to the cone bearing values

Site	Condition of pavement	Existing thickness in.	Thickness required from McLeod's chart in.	Calculated thickness in.
28/7	Cracked	11.5	18	17.5
34/3	,,	11.5	17	10.5
40/4	,,	11.5	6	5
48/2	,,	15	8	7
54/2	,,	12	6	4.5
31/3	Sound	13	16	10
42/5	,,	15	10.5	7.5
59/1	,,	14	6	6
60/7	,,	21	7	6.5

It is seen from column 4 of Table 10 that the pavements at miles 28/7, 34/3 and 31/3 are of inadequate thickness when considered from the point of view of the thickness chart; but, only M 28/7 is of inadequate thickness from the point of view of the calculated thickness from the formula. The latter is in conformity with the earlier observations. The fact that M 31/3 is sound and that M 34/3 is of adequate thickness, as seen from the values of C.B.R. and plate bearing, shows that the interpretation with cone-bearing values is not very consistent.

Photo 1 shows the cracked bituminous road surface in M 28/7 which has developed map-pattern or alligator type cracks. Besides the greater width of the cracks, the loops are formed parallel to the line of traffic. Generally, these are the characteristics of road surfaces that fail due to a weak subgrade. Also, the cracks develop right down to the subgrade.

Photo 2 shows a typical sound bituminous road surface at M 42/5, where the pavement is constructed of adequate thickness.

3.3.2. Test Sections in the Widened Portion of the Road

Considering the test sections in the widened portion of the road, which are all about 7 in. thick without exception, it may again be seen that some have developed cracks while some have not. From an analysis of the results of the subgrades at these sites, similar to that of the sites at the original portion of the road, using the same curve for 9000 lb wheel load in Fig. 6, it was seen that some of the sound sections also lacked the adequate thickness as dictated by this curve. Hence an analysis based on the curve for 7000 lb wheel load was adopted for reasons stated earlier, viz., that the intensity of traffic on the widened portions is not the same as that in the centre, and this yields the data given in Table 11.

TABLE 11

Required thickness of pavement

Site	Condition of pavement	Existing thickness	Thickness from McLeod's chart for			Calculated thickness
			C.B.R.	'K'	Cone bearing	
		in.	in.	in.	in.	in.
28/7	cracked	7	17	16	12	10
31/4	do.	7	14	12	—*	
42/5	sound	7	6	6	10	10
48/2	do.	7	6	11	6	6
52/7	do.	7	6	6	6	5
60/7	do.	7	6	6	8	8

It is evident from Table 11 that the sites at miles 28/7 and 31/4 have failed distinctly due to an inadequate thickness of the pavement, when considered from any of the tests, viz., the field C.B.R., plate bearing and the cone bearing.

Considering the sound sections, it may be seen that though all of them have the necessary thickness from the field C.B.R. point of view, M 48/2 does not satisfy the requirements based on the plate bearing test and miles 42/5 and 60/7 do not satisfy the

* Saturated subgrade slumped under test.

cone bearing test, which demands a higher thickness than the existing one in all the three cases. However, considering only the plate bearing test values which are more consistent than the cone bearing data, the reason for the soundness of the road at M 48/2 may be attributed to the possibility of the subgrade soil not having been saturated in actual performance to such an extent as to lose its strength below 370 p.s.i./in., which corresponds to a thickness of about 7 in.

Photo 3 shows the surface cracks on the bituminous road in the widening at M 31/4. The cracks are in a fairly advanced stage, followed by visible settlement of the surface, which is indicative of the fact that the failure is due to a weak subgrade. The uncompacted nature of the subgrade at this site was further revealed when the subrade in a portion of the test pit slumped during saturation, Photo 4.

A typical sound road surface in the widened portion pertaining to M 48/2 is shown in Photo 5.

3.3.3. Summing up, it may reasonably be concluded that the failure of the road in miles 28/7 and 31/4 is definitely due to inadequate thicknesses of pavement, whereas the failures in miles 34/3, 40/4, 48/2 and 54/2 are not due to deficiency in the pavement thicknesses but due to other reasons discussed below.

3.4. Failures due to Subsidence of the Base or Wearing Course Aided by Weathering of Binder.

The causes of failure in a pavement, constructed with adequate thickness, should normally be located in the base and wearing courses due to one or more of the following reasons :

1. Inferior type of materials in base and wearing courses;
2. Non-homogeneity of the materials used in the base and wearing courses;
3. Inadequate compaction of the materials in base and wearing courses;
4. Inadequate control in the construction, of the various courses
5. Weathering of the binder in the wearing surface.

Hence, in order to trace the causes of failures in the sites at miles 34/3, 40/4, 48/2 and 54/2, which have adequate pavement

TABLE 12

Aggregate impact values

No.	Material	1st zone (M29—M35)		2nd zone (M36—M48)		3rd zone (M49—M61)	
		Dry per cent	Wet per cent	Dry per cent	Wet per cent	Dry per cent	Wet per cent
1	Stone metal	19.60	15.62	11.62	15.74	18.28	18.57
2	Kankar	29.20	29.34	10.94	17.03	15.66	17.29
3	Brick	37.30	39.00	52.27	51.60	31.40	27.40

TABLE 13

Field C.B.R. and plate bearing tests on kankar base

No.	Location (original)	Condition of pavement	Thickness of kankar base in.	Field C.B.R. values		Plate bearing 'K' p.s.i./in.	
				At existing M.C. per cent	After saturation per cent	At existing M.C.	After saturation
1	M31/3	sound	10-12	—	—	1540	960
2	M34/3	cracked	9-10	—	—	880	860
3	M54/2	cracked	8-9	128	95.0	—	—
4	M59/1	sound	10-11	—	101.3	—	—

thicknesses, it is essential to study the characteristics of the base and wearing courses.

Analysing the materials of the base and wearing courses, it will be seen from Table 12 that the kankar used in the base course and the stone metal in the wearing course are sufficiently strong.

A few sample tests of plate bearing and C.B.R. done on the kankar bed in a few places, Table 13, reveal that even after saturation the values of plate bearing and C.B.R. are as high as 860 p.s.i./in. and 95 per cent respectively, thereby showing that the base course is adequately strong.

Considering the characteristics of the binder in the wearing surface, it is noticed from Table 14 that the E.V.T. (Equi Viscous

Temperature) and the softening point of the extracted binder are higher than the standards laid by the I.S.I. [7] This obviously shows that the lighter fractions of the tar have evaporated leaving a more viscous residue which has a higher E.V.T. and softening point. Under very cold climatic conditions and with age this material tends to become very brittle.

TABLE 14

E.V.T. and softening point values of the extracted binder from the bituminous wearing surfaces

No.	Extracted sample from	E.V.T. °C.	S.Pt. °C.
1	M 28/7	97.5	87
2	M 31/3	90	59
3	M 31/4	76	42.5
4	M 34/3	78	30

Though evaporation and oxidation of the tar by themselves need not always lead to cracks on the bituminous surface, the binder becomes too brittle to adjust to the state of stress in the pavement when tensile forces are set up on the wearing coat by the passing traffic, thereby resulting in cracks on the bituminous surface. Such cracks due to weathered binder appear invariably in all bituminous roads in the course of time where the surface is not treated subsequently in any manner. These cracks appear first as fine hair cracks and are subsequently aggravated by traffic and ingress of moisture.

Of the four failed sites, viz., miles 34/4, 40/4, 48/2 and 54/2 having adequate thicknesses, the failure in M 48/2 appears to be due to a weathered binder, considering that there was no visible settlement of the road surface and that the cracks are extremely fine, Photo 6, which could be easily distinguished from those in miles 34/3, 40/4 and 54/2. Also, the depth of the cracks is limited to the wearing surface only. The latter three sites fall into one group, considering either the thickness or the C.B.R. and plate bearing values of the subgrade soil, whereas the site at M 48/2 seems to be different. Hence, it is reasonable to isolate the cause of failure of the site at M 48/2 from those in miles 34/3, 40/4 and 54/2, which is fully supported by Photo 6.

The surface cracks at miles 34/3, 40/4 and 54/2, which have developed almost on the whole face of the road, could not be attributed solely to the weathering of the binder, since the cracks are of a definite pattern and they are not very thin. Besides, they have penetrated into the wearing course also in some places. Hence, to pin point the causes of cracks in these sites, it is necessary to analyse the phenomenon of the development of cracks on an otherwise sound road.

When there is adequate thickness of pavement and the base and wearing course materials are reasonably sound and if failures should still occur in the form of extensive cracks, it is definite that the failure should be only due to the non-uniformity of construction or non-homogeneity of materials in the base and wearing courses when the binder in the wearing surface has lost its plastic properties. Under these conditions, any local weak spot in the wearing or base course is likely toyield under traffic, with in itself and may be termed as a 'subsidence'. The weak spot may be either due to the non-homogeneity of the materials of the wearing or base course or due to lack of proper uniform compaction during construction or due to the ingress of moisture. Since the binder has lost its flexibility, the subsidence is followed by cracks. One such typical crack due to subsidence of the base or wearing course is seen in Photo 7 on an otherwise sound road surface, which is the first stage in the development of such cracks. Since the area around this crack is weakened, further cracks develop around the first crack due to passing wheel loads which produce a tension zone on either side of a crack, Fig. 14. Due to the weathered nature of the binder, the wearing

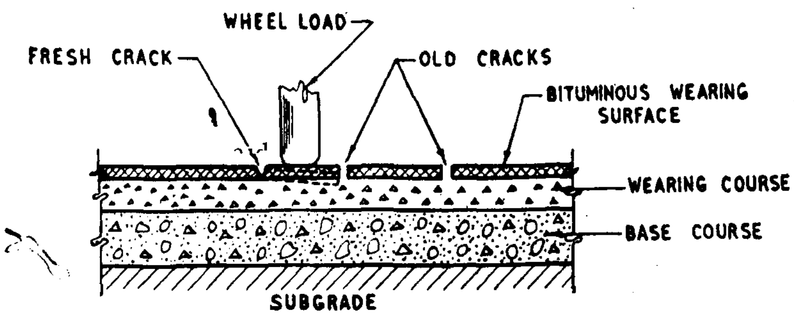


Fig. 14. Development of cracks due to wheel loads

surface cannot adjust itself to the state of stress in it and develop cracks adjacent to the previous cracks.

Further, any moisture that enters the pavement through these cracks will weaken the contact plane between the wearing coat and the layer below, as a result of which further cracks will develop around the earlier cracks which may be described as the second stage in the formation of cracks, Photo 8. In the final stage the cracks extend to the whole width of the road in the course of time, Photo 9, and pitting of the surfacing will take place, leading to the end of its life, Photo 10.

The notable feature in the sites at miles 34/3, 40/4 and 54/2 was that there were no visible settlements on the road even though the surfaces were marred completely with cracks, Photos 11 & 12. However, it is difficult to distinguish the difference in the cracks of this type from those due to settlement of the subgrade, because in the former case cracks are bound to develop parallel to the line of traffic in the final stages as much as in the latter case; yet, the thickness of the cracks is a distinguishing factor as found in Photo 12 where it may be seen that the width of the cracks is comparatively thin even though the pitting has occurred on the surface, and also the depth of the cracks is limited to the wearing course. But the cracks which develop as a result of the weathering of the binder alone are distinctly different from others in that they are extremely thin and their loops have no relation to the direction of the movement of the traffic.

In the light of the above factors, it appears probable that the pavements at sites M 34/3, 40/4 and 54/2 having good subgrades and adequate thicknesses might have developed cracks as a result of subsidence of the base or wearing course, aided by the weathering of the binder.

3.5. Failure Due to Waterlogging

The test site at M 50/2 is a typical case of the stretch of the road from miles 50 to 52 which lies in a very low lying waterlogged area where water is stagnating on either side of the road all round the year and the water table is hardly about 8 ft below the wearing surface. Heavy settlements occurred all along this stretch and the road surface was full of deep corrugations and cracks. It was gathered that during rainy season floods come right over the wearing surface.

It may be seen from the plasticity chart that the subgrade is of CL soil and that the existing moisture content is as high as 16.28 per cent. Hence, it is not surprising that even at the existing moisture content the plate bearing value is only 300 p.s.i./in. No tests were done at this site after saturating the subgrade because the latter became almost a slurry on saturation, rendering the site unsuitable for tests. Since M 50/2 is a clear case of a road in a waterlogged area, and the subgrade is a CL soil, severe plastic deformation of the subgrade under the wheel loads occurred which, in turn led to heavy settlements of the pavement. In fact, it was observed that in many places along this stretch of the road in waterlogged area the road bed had sunk to quite a good depth and the whole surface of the pavement was full of cracks. One such typical cracked surface is seen in Photo 13.

Hence, the failure of the road bed at M 50/2 may primarily be attributed to the failure of the subgrade due to plastic deformation arising from waterlogged conditions

3.6. Development of Cracks on Patchworks

It was found during the investigation that in some places cracks had appeared on the patchwork which was laid on a cracked bituminous surface. Photo 14 shows the cracks on the fresh patch-work in M 50/2. It was noted that the binder in the patchwork was quite alive. Obviously, the cracks on the patchwork are due to the settlement of the clayey subgrade which sets up very high tensile stresses in the pavement. Hence, it is felt that any amount of patchwork will not help so long as there is an inadequate thickness of pavement on a subgrade which is weak and settling.

3.7. Hair Cracks in the Subgrade Soil

A peculiar occurrence, namely, the presence of hair cracks in the subgrade soil, was observed in the sites at M 34/3, 40/4 and 59/1. The hair cracks had developed in a distinct pattern as can be seen from Photos 15 & 16, the former depicting the cracks in the natural state and the latter showing them marked by a pencil for a more clear presentation.

Whereas the surface cracks are marked by their width and depth quite visible to the naked eye, the so-called cracks on the subgrade soil are conspicuous only by the dark coloured lines on the subgrade. The lines, which are due to the excess moisture which occupies the cracks, gradually disappear on evaporation if the subgrade surface is exposed to the atmosphere for some time and can be seen again if the dried surface is scraped off.

It will be seen from Tables 6 and 7 that the hair cracks had occurred only in places where the subgrade soil was predominantly sandy in nature. The fact that in M 34/3 and 40/4 the bituminous wearing surface was cracked and that in M 59/1 it was sound without cracks indicates that the hair cracks in the subgrade were not influenced by the surface cracks.

Photos 17 & 18 clearly show that the subgrade cracks have penetrated vertically down to some depth and then gradually disappear. The high sandy nature of the soil at these places suggests that the cracks might have been formed by the differential densification resulting in an elastic deformation of the sand under the base course. To produce this effect, the base course should have had cracks in it and should exist as a block rather than as a solid single sheet.

In all these test sites, the cracks were observed on the subgrade under the kankar base and not under the brick soling. However, it is felt that the hair cracks may develop under the brick soling also in the course of time under similar traffic and subgrade soil conditions, if the bricks are not crushed during the process.

From the high values of C.B.R. and plate bearing test on the subgrade soil with hair cracks, it may be inferred that the presence of the hair cracks in the subgrade soils need not be harmful to the structure of the road bed as such.

4. CONCLUSIONS

4.1. From the limited data obtained in this investigation some broad conclusions can be drawn as pertaining to the interpretation of the test values in general.

However, it may be borne in mind that extensive investigations over a wide range of soil and climatic conditions would be necessary before the conclusions set forth below can be taken as universal, but from the limited investigation carried out and as reported in this Paper the following conclusions are indicated:

1. Just as a relationship seems to exist between the static plate bearing test values and the laboratory C.B.R. data, a very broad relationship appears to exist between the 'K' values and the field C.B.R. values. However, even this relationship is valid only over a very limited range.
2. Using the above relationship, the values of the thickness of pavement required for different values of the static plate bearing test can be obtained based on either the conventional laboratory C.B.R. curves or McLeod's field C.B.R. curves.

3. The thickness of a pavement required for a given wheel load is less if based on field data (whether 'K' values or CBR) than that obtained from the laboratory CBR values and as such the designs based on the latter are more conservative.
4. The static plate bearing values using a 12 in. diameter plate can be utilized for the design of the thickness of flexible pavements in the same manner as the laboratory C.B.R. or field C.B.R. or the repetitive plate bearing tests using the appropriate curves.
5. An analysis of the performance of the pavement based on the North Dakota Cone does not appear to be so very consistent as an analysis based on 'K' or C.B.R. values.
6. Proctor's penetration value as a test procedure for the evaluation of pavement thickness and performance is not wholly reliable.

4.2. As regards the causes for the extensive surface cracks over the forty mile stretch that was investigated, the following are the main conclusions:

1. The presence of surface cracks in the sites at miles 28/7 and 31/4 can be mainly attributed to weak subgrades or, in other words, to inadequate thickness of pavements.

   The settlement cracks are very wide and are of alligator or map pattern, followed by surface settlements; the loops are formed mostly parallel to the direction of the movement of traffic and the cracks develop right down to the subgrade.

   The presence of cracks on the bituminous wearing surface at miles 34/3, 40/4 and 54/2 appears mainly to have emanated from the failure of the wearing or base course, aided by the weathering of the binder.

   It is difficult to distinguish the cracks due to subsidence of the base or wearing course in the final stages from those due to settlement, as in both cases the cracks are of map pattern. However, the depth of the cracks due to local subsidence does not always travel right down to the subgrade and is limited to the wearing course. This, together with no visible settlement, differentiates the failures.

   The cracks at M 48/2 appear to have their origin solely in the weathering of the binder in the surface course.

The surface cracks due to weathering of the binder are distinguishable from other cracks in that the former are very thin followed by no surface irregularities, and that the loops do not have any relationship with the direction of the movement of traffic. Also, the depth of the cracks is limited to the wearing surface only.

4. In the site at M50/2, the failure of the pavement is due primarily to the waterlogged condition of the road bed founded on a clayey subgrade soil.

5. Any amount of bituminous patchwork will not help in removing the cracks from the existing road, as long as the thickness of the pavement is inadequate to carry the heavy wheel loads.

6. The presence of hair cracks in subgrades of sandy and non-plastic soils is believed to be due to the differential densification of the sand. It is felt that the presence of these hair cracks is not harmful to the structure of the road as such.

5. RECOMMENDATIONS

Based on the results of this investigation, the following :commendations can be suggested:

1. Periodical renewal of surface dressings is necessary at regular intervals of time so that the surfaces may not get cracked as a result of starvation and weathering of binder. The interval between two renewal courses would depend on the local conditions of traffic, etc.

2. In the cases where failure of the pavement is due to inadequate thickness of the road structure, a programme of progressive building of the road bed may be drawn and executed so as to meet the demands of the traffic conditions. This building up can be either in the form of multiple surface treatment or by having thicker premix carpets.

3. In the case of renewal courses over cracked surfacings, if the surface is pitted considerably, complete scarification of the surface should be done for laying the new coat. Just resorting to heavy patching over a cracked surface will not always help.

4. It is suggested that greater care should be ensured during the construction of the water-bound macadam itself, such as proper control over the quality of the material that is being used for the construction.

5. In the case of road beds in waterlogged areas, detailed investigations will have to be carried out for different cases so that adequate provisions for preventing failures of the road structure as a result of flooding may be made.

ACKNOWLEDGMENT

The Authors wish to thank the Director of the Central Road Research Institute, New Delhi, for his permission to publish this Paper. They also wish to record the help rendered by Messrs R. L. Nanda, S. R. Bindra, K. P. Nair and R. S. Shukla in conducting the various tests in the field and in the laboratory.

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TABLE 15

Abstract of traffic census taken from 8 A.M. on 22-2-57 to 8 A.M. on 23-2-57

S. No.	Type of traffic		No. of vehicles (up and down)	Wt. of each vehicle in tons	Total traffic in tons
1	Lorries-loaded	...	281	6	1686
2	Lorries-empty	...	30	2	60
3	Passenger buses	...	136	5	680
4	Private cars	...	129	1.5	194
5	Motor cycles	...	5		
6	Bullock artes & tongas loaded	...	64	1	34
7	Bullock carts & tongas empty	...	27	0.5	14
	Military vehicles:				
8	Lorries	...	8	8	64
9	Cars	...	2	1.5	3
10	Other vehicles	...	3	1	3
11	Cycles	...	339		
			Total	2768 : Say :	3000 tons

Paper No. 212

"LANDSLIDE INVESTIGATION AND TREATMENT IN ASSAM"

By

G. C. SHARMA,* M.Sc. (ENG.), C.E. (IOWA)

CONTENTS

SYNOPSIS

This Paper describes the landslide problems in the State of Assam in general and mainly on Dimapur-Imphal Road, National Highway No. 39. Methods adopted so far to control landslides are described and suggestions are made to prevent their occurrence.

Present day information regarding the theory and causes of landslides and subsidences, investigation procedures with the knowledge of soil mechanics and geology, analysis of soil studies and assessment of correct solutions of individual problems faced by highway engineers have, as far as possible, been incorporated in the Paper.

1. INTRODUCTION

1.1. Highway engineers have to face acute problems when they have to deal with landslides and subsidences which are of common occurrence in the construction of hill roads. A large sum of money is spent every year to eliminate and control these troubles, but, usually, anticipated results are not achieved com-

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mensurate with the expenditure incurred. In a country where mountainous regions are vast, rainfall intensity and concentration are high, seismological disturbances are frequent, the geological structure is young and unstable, the problems of landslides and subsidences are very complex and, as such, thorough investigations and their correct analysis and assessment are necessary to decide upon any preventive or control measures that have to be adopted.

1.2. In Assam, landslides occur frequently and the problems are very common in the hilly districts of Khasi and Jaintia Hills, Mizo Hills, Garo Hills, North Cachar and Mikir Hills, and severe in the case of Naga Hills. There are recent instances of heavy casualties of human beings besides losses of livestock and other private property due to very heavy and sudden landslides in a wide area in the North-East Frontier Agency adjacent to the State of Assam. Annexure 1 will indicate the expenditure incurred both for original and repair works during the last five years in dealing with a few landslides on the Dimapur-Imphal road.

1.3. A complete and thorough search of each landslide problem in Assam has not yet been taken up. It is expected that a full scale study will be made in each case in the near future with the help of a soil testing laboratory. In this Paper, studies of Dimapur-Imphal road, which is badly affected by landslides, have been made based on geological and other considerations of some of the soil types. Corrective measures since adopted have indicated some measure of success to warrant further investigations and remedies on the same lines.

2. THEORY AND CAUSES OF LANDSLIDES AND SUBSIDENCES

2.1. Men have been suffering for thousands of years directly or indirectly due to landslides but, on account of their limited knowledge and lack of technical advancement, the way of solving such problems was not clear for a long time. Since the last [illegible] years, engineers have directed their attention to these studies in a scientific manner and have been thereby successful in decreasing the magnitude of hazards involved to human life. Such studies are, however, unable to give quantitative answers to the problem of prevention and correction of highway landslides and subsidences. Hence, there is a need for field investigation and laboratory analysis for each and every individual case to deal with it in the right direction.

2.2. According to Terzaghi, landslides may be defined as "the rapid displacement of a mass of rock, residual soil or settlement adjoining to a slope in which the centre of gravity of the moving mass advances in a downward and outward direction." He has classified landslides and details of mass movements into two main divisions, viz., (1) dry movement with static friction fully effective, (2) movement with static friction wholly or partly eliminated. These classifications can again be subdivided into the following: (i) slow flowing, (ii) rapid flowing, (iii) sliding, and (iv) subsidence or sinking of ground to an appreciable depth.

2.3. The main physical factor of earth movement is the reaction of stresses induced in soil particles which are under the influence of the gravitational forces. A mass of rock, when detached from the bed rock, cannot remain static unless unusual means are adopted to balance the mechanical equilibrium. Similarly, if the underside of the soil which is inclined to the horizontal plane is lubricated, it cannot but move downwards along the inclined plane due to reduction of shearing strength of soil particles. If that tendency is not counterbalanced by shearing resistance, the soil mass is sure to slide. Minor earth movements are due to the results of instability of earth crust and are mainly subject to laws of mechanics.

The assessment of forces causing intergranular movements is a very important factor. Coulomb's law of friction is expressed by the formula:

$$F = \mu P \qquad \text{(1)}$$

where μ = coefficient of friction.

F = frictional resistance.

P = normal pressure.

The coefficient of friction depends on many factors in the case of soils, such as moisture, temperature, surface texture, etc. Slides in natural soil are mainly due to disturbances by cutting slopes, digging holes, and excavations with unsupported sides. From the physical behaviour, the landslides may be classified into the following categories:

(1) Natural slides in heterogeneous soils due to uplift acting on impermeable strata underlain by saturated permeable stratum, sudden consolidation of a saturated sandy strata and joints, bedding planes or faults.

(2) Natural slides in homogeneous soils due to internal friction, cohesive properties of soils and factor of safety against the sliding force.

(3) Sliding due to plastic flowage.

(4) Breaking away of overhanging strata and subsequent collapse of the soil mass.

2.4. According to Terzaghi, the causes of landslides may be ivided into two main classes, viz , (1) external and (2) internal. 'he common reasons for external causes are: (i) decrease in upport at the foot of a slope and increase in weight on supermposed edge of the slope and (ii) earthquake shocks which ncrease the shearing stresses of the soil masses along the sliding lane. The internal causes are mainly due to the action and nfluence of water which can be divided into (i) loss of cohesion of soil particles, (ii) increase of pore water pressure and decrease of shearing stresses, (iii) seepage, (iv) spontaneous liquefaction, and (v) seepage of water from artificial sources which reduces the shearing resistance of soil by increasing water pressure. The main phenomena hastening the physical processes of landslides are summarized briefly as (i) steep slope of catchment, (ii) friability of soils, (iii) ground water action such as seepage, saturation, etc., (iv) heavy rains or melting snow, (v) deforestation, (vi) lack of ecological observations, (vii) seismic actions, (viii) erosion by rivers and torrents at the toe of the slope, and (ix) swamp and waterlogging in the area.

2.5. Slumping is the downward slipping of a mass of rocks or unconsolidated materials moving as a unit or subsidiary unit with more or less horizontal axis parallel to the slope. It occurs in unconsolidated detritus of old landslides composed of loam mixed with fresh water, limestone and peat. Slumps are most abundant where cliffs are relatively low.

3. INVESTIGATION PROCEDURES

3.1. Maintenance of Records

In order to investigate a landslide problem, suitable and authentic records should be available. Therefore, a register should be maintained which will indicate the following information:

(1) Time and year when the landslide is first noticed with full history.

(2) All meteorological information such as rainfall precipitation, temperature, humidity, period of drought, freezing and thawing period, if any, after severe winter, etc.

(3) All previous measures adopted to stabilize the landslides such as piling with spacing, depth and number of piles, retaining and breast walls, R.C. cribbing, etc., with complete design data as available.

(4) Records of similar slides in the vicinity with protective measures adopted.

(5) History and date of earthquake with intensity.

(6) Area if cultivated, forest reserve, deforestation, jhoom cultivation, terraces, nature of vegetation grown in the area.

(7) Presence of mines, quarries, etc.

(8) A topographical or contour map of the area showing the location of the slip area, existing roads, old roads damaged or subsided, rivers, springs, lakes, catchment drains, the adjacent ground and new landslides, etc.

(9) Full descriptions stating the nature, type, etc., of the bed rock with petrography and mineralogy of underlying soils with their conditions in nearby areas.

(10) Geophysical survey data such as location of slip surface, soil conditions relating to the moisture, density, structure, physical properties, direction and type of movements, their duration, speed, and so on.

3.2. Geological Log

A geological log prepared by an experienced geologist who is well acquainted with the vicinity is vitally important. It gives the general descriptions of geological formations and irregularities of the area which help the highway engineer to locate the highways or to deal with the landslide problems.

3.3. Soil Samples

3.3.1. Firstly, a visual reconnaissance should be made for preliminary investigation by dividing the whole area into zones according to the nature of slides occurred. Stacks should be driven in each zonal area where soil samples are to be collected. Such stacks should be numbered and connected with the centre line of the road by stations, offset distances, and surface elevations connected with several permanent bench marks. The zones should be selected in such a way that some stacks will fall near the top, some away down in the side slopes and some near the toe of the slope, care being taken that the whole area under investigation is fully covered.

From each of the zonal areas and from each point, soil samples both disturbed and undisturbed should be collected and sent to the laboratory for finding out the water content, mechanical analysis, tests of consolidation, shearing strength, density, etc. The drill data will also show the underlying bed rock or stable soil layers to determine the slip surface or slipping plane and also the seepage line, ground water line, water table, etc.

3.3.2. Brief notes on the physical properties and behaviour of each type of soil in landslide areas are furnished below which may help the engineers to locate the highway alignments.

(1) **Granular type of soil:** Soils in cutting or embankment consisting of sand, gravel and rock particles seldom involve sliding.

(2) **Silt:** Silty soil may slough when not compacted properly. It may be easily eroded when the soil is fully saturated. Silt also seldom slides unless a clay, shale or rock slipping plane is established.

(3) **Clay**: Clay is a dangerous type of soil always susceptible to sliding and instability when such clays are characterised by high liquid limit and high plasticity index which can be ascertained from the laboratory tests. Expansive clays and organic silts may be considered as most unsuitable material for embankment unless specially treated. .

(4) **Shale**: Shale by nature is a stable material, but when used in road construction, it presents a potential landslide material.

(5) **Rock with clay or clay shale**: When cuts are made in the outer slopes, the shale is unusually found to be disintegrated by weathering, forming a plastic clay. Such materials are very difficult to compact due to excess voids and when these voids are filled up with water, sloughing and sliding occur.

3.4. Cross Section

3.4.1. To ascertain the velocity and movement of soil or rock masses, there is no surer way than to take a series of cross

sections of the whole landslide area. These cross sections should be taken at the time of first appearance of a landslide. The old constructional records should be obtained and new sections superimposed over them. From the comparison, foundation failure, side hill fills, slipping plane, settlement, sloughing, etc., can be judged. In all cross sections, the following should be plotted :

(1) All borings or drill informations with depth of soil strata, etc.

(2) Results of laboratory tests on soils such as mechanical analysis, plastic limit, liquid limit, plasticity index, field moisture content, density, triaxial compression test data, etc.

(3) Data pertaining to the soil layers and underlying bed rocks with mineralogical test results such as X-ray, differential thermal, petrographic and weathering conditions, etc.

(4) Surface cracks and cavities with depths and sizes of structures and other tension cracks.

(5) Ground line both before and after the movement.

(6) Ground water table or seepage line and any other hydrological information.

3.5. Theoretical Analysis

3.5.1. ***Stability of slopes***: The stability of slopes depends on many factors such as height of slope, unit weight of soil, steepness or angle of the slope, cohesion, internal friction and action of ground or seepage water.

3.5.2. There are several theories and methods concerning stability of slopes and the commonest ones are given below. The details of each method are available in any text book on soil mechanics and hence they are not fully discussed here. The various methods are :

(1) Culman method.

(2) Risal-Frontard method.

(3) Circular arc method.

(4) Slice or Swedish method advanced by the Swedish Geotechnical Commission.

(5) ϕ Circular method developed by Professor Glennon Gilbay and Arthur Casagrande.

(6) Jacky method.

(7) Logarithmic spiral method.

(8) Praudtl's method.

(9) Terzaghi's method.

3.5.3. Almost all the methods are based on the assumption that soil is homogeneous, isotropic, of constant angle of slope and shearing strength based on Coulomb's empirical equation :

$$S = C + N \tan\phi \qquad \ldots\ldots\ldots\ldots\ldots \quad (2)$$

where S = the unit shearing strength.

C = the unit cohesion in pounds per square foot.

N = normal stress on the surface.

ϕ = the effective angle of internal friction.

In a sliding area, two forces work together, viz., the activating force and the opposing or balancing force. When the movement of the activating force overcomes that of the balancing force, the slide occurs along the critical surface or slipping plane. This can be expressed as :

$$\Sigma P = CL + \Sigma N \tan\phi \qquad \ldots\ldots\ldots\ldots\ldots \quad (3)$$

where ΣN = summation of normal forces in pounds.

ϕ = angle of internal friction in degrees.

C = cohesion in pounds per square foot.

L = length of the slip surface in feet.

ΣP = summation of tangential forces or shearing forces.

From the above, the factor of safety (F.S.) for sliding of the slope may be deduced as :

$$F.S. = \frac{\text{Moments of forces which oppose sliding}}{\text{Moments of forces which activate sliding}}$$

$$\text{i.e., } F.S. = \frac{\Sigma N \tan\phi + CL}{\Sigma P} \qquad \ldots\ldots\ldots\ldots\ldots \quad (4)$$

The value of 'C' and 'ϕ' may be determined in the laboratory by shear test or by unconfined compression test.

When facility for such testing of soils is lacking, the following table may serve for approximate analysis :

TABLE 1

Type of soil	C in lb per sq. ft.	ϕ in degrees
Almost liquid	100	0
Very soft clay	200	2
Soft clay	400	4
Medium clay	1000	6
Stiff clay	1500	8
Muddy sand	400	30
Very dense sand and gravel	1000	34

If the computed factor of safety exceeds unity, the slope can be considered as somewhat stable. But there are other forces affecting stability. To be on the safe side, it is considered necessary to take F.S. more than unity. For natural slope analysis, undisturbed soil samples are to be tested.

3.5.4. When hydrostatic pressure is in existence and requires to be accounted for, the following formula may be applied :

$$\Sigma P = CL + \Sigma (N - \mu) \tan\phi \qquad \ldots\ldots\ldots\ldots\ldots \quad (5)$$

where μ = $hrw\, l$ = water pressure in pounds at the slipping plane.

h = depth in feet from ground water level to the slipping plane.

rw = unit weight of water = 62 lb per cu. ft.

l = length of slip surface in feet along the slipping plane.

$$FS = \frac{CL + \Sigma (N - \mu) \tan\phi}{\Sigma P} \qquad \ldots\ldots\ldots\ldots\ldots \quad (6)$$

3.5.5. Experience of many years establishes that the plane of failure is a curve steep at the top and flattening out at the bottom. Because of its simplicity and flexibility, the ϕ circle method developed by Krey in Germany and elaborated by Dr. Terzaghi has widely been adopted. Professor Taylor of the Massachussetts Institute of Technology has proved by analysis, as may be seen from the following Table, that the results obtained from Fellenius' general solution by graphics using slice method are slightly higher than the results based on mathematical solution on ϕ circle method. He has shown that for all practical purposes, the results obtained by ϕ circle and logarithmic spiral method are identical. It is also shown that all the three methods become identical when ϕ = zero.

TABLE 2

COMPARISON OF VALUES OF C/FWH BY SLICE AND ϕ—CIRCLE METHOD

Slope angle	$\phi = 5$		$\phi = 15$		$\phi = 25$	
	Slice	ϕ Circle	Slice	ϕ Circle	Slice	ϕ Circle
15	0.72	0.70	—	—	—	—
30	0.114	0.110	0.048	0.046	0.012	0.009
45	0.141	0.136	0.085	0.083	0.048	0.044
60	0.165	0.162	0.120	0.116	0.082	0.074
75	0.196	0.195	0.154	0.152	0.118	0.117
90	0.239	0.239	0.199	0.199	0.165	0.166

He has worked out some graphs by which the stability of slope can be found out by use of stability number when the slope angles and the friction angles are known, without going through complicated mathematical calculations. The stability number is related to other variables as it appears from the following equation:

$$S = \frac{C}{FWH} \qquad \ldots\ldots\ldots\ldots\ldots \quad (7)$$

where S = stability number.
C = cohesion of soil.
F = factor of safety.
W = unit weight of soil.
H = height of slope.

3.5.6. For practical purposes, the graphical analysis based on slice method as described below is considered suitable for

analysing landslides with natural slopes when soil conditions are variable. Failure surfaces or slip surfaces may be plotted in the following manner: The starting of slipping plane may be ascertained from the open crack or cavity or surface displacement at the bottom, a few feet away from the toe, before rupture takes place, Fig. 1. The curve should start beyond the crack or cavity which is formed due to tension and should be taken into

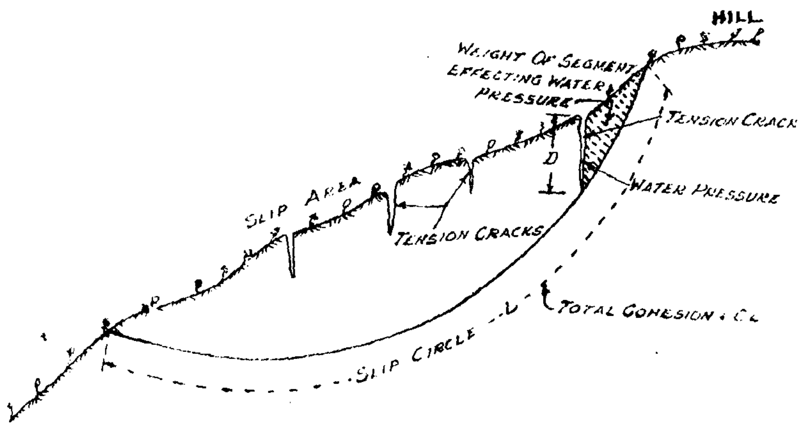


Fig. 1

consideration in stability analysis. The inclusion of the entire weight of the portion takes care to some extent for possible hydrostatic pressure due to water filling in the tension cracks. The depth of tension cracks may be worked out from the following formula established by Terzaghi:

$$D = \frac{2C \tan (45^\circ + \phi / 2)}{r_b} \qquad \ldots\ldots\ldots\ldots\ldots \quad (8)$$

where D = depth of tension crack in feet.
C = cohesion in pounds.
ϕ = angle of shearing resistance.
r_b = bulk density of soil in pounds per cu. ft.

In the case of purely cohesive soils or soils with small shearing resistance, the formula used is:

$$D = \frac{2C}{r_b} \qquad \ldots\ldots\ldots\ldots\ldots \quad (9)$$

If the original ground surface cannot be ascertained, that may correctly be assumed from the nature of the slope in that area or nearby vicinity. In each cross section, a series of borings may be made and plotted therein showing the results of laboratory soil tests data, seepage strata, location of underlying bedrock, surface cracks, cavities and structures, etc. In these cross sections, the theoretical curves worked out by formula as well as curves based on boring data should be plotted, Fig. 2. Then the

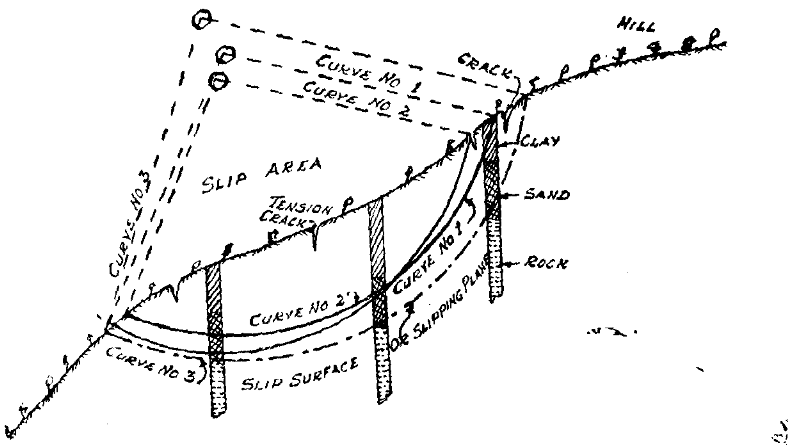


Fig. 2

curves which show the worst condition may be taken into consideration for theoretical stability analysis.

3.5.7. After drawing the critical slipping plane, the cross section is divided into a series of increments of equal width parallel to the direction of the moving force. The weight of each increment to a width of one foot is computed and then the resolutions of forces are graphically worked out as described below and shown in Figs. 4 and 5. By doing so, the total normal forces and shearing forces acting in the sliding area are obtained. The other two variables such as C (cohesion) and ϕ (angle of internal friction) may be obtained from the shear test or unconfined compression test of soil samples in the testing laboratory.

3.5.8. When boring facilities in the sliding areas are not easily available and when early estimate of maximum depth of the slide from the ground surface to the slipping plane or plane of rupture is required, the following method known as "slip circle method" as devised by Arthur M. Ritche may be adopted.

The points A and B may be ascertained from the physical condition of the slipping block plotted to scale showing the ground line as well, Fig. 3. Position of A may be obtained from the inspection of tension cracks and B from the toe of the slipping mass. Join AB and bisect it at point C. Through C and A draw the perpendicular bisector and a horizontal line

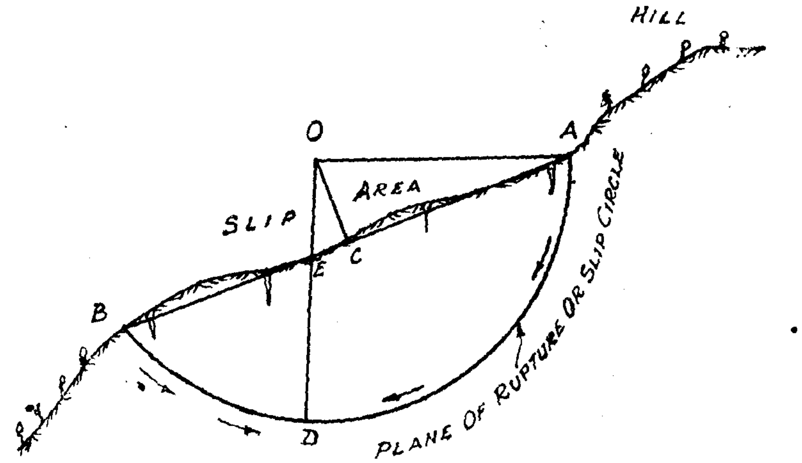


Fig. 3

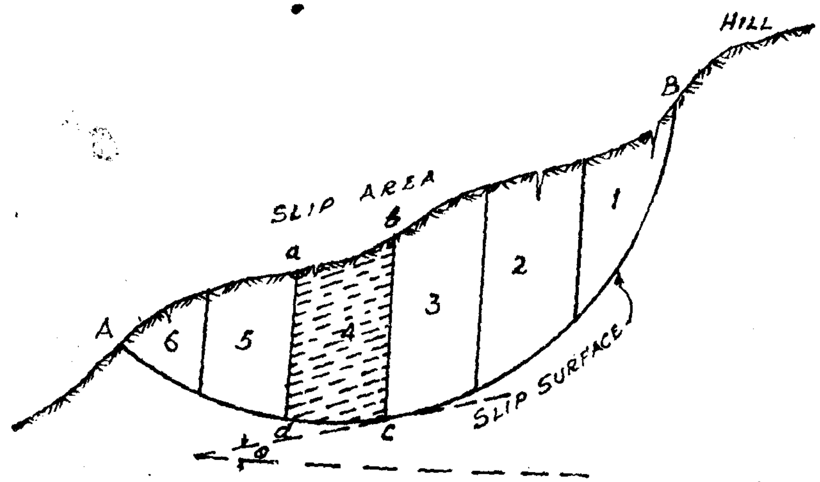


Fig. 4

meeting at point O. The point O should be in such a position that the tangent of the line through A to the slip circle is never steeper than vertical. With the radius OA, draw an arc. Then the maximum depth may be obtained from the line OD at point D.

3.5.9. The slipping or critical surface AB is drawn as shown in Fig. 3 and the cross section is divided into a convenient number of equal vertical slices. All the slices are taken to be of unit thickness. As an example, the stability of any strip *abcd*

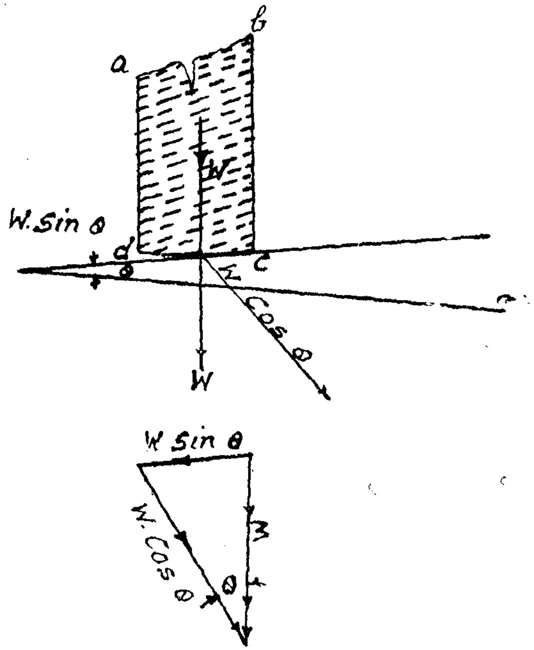


Fig. 5

in Fig. 4 has been analysed by resolution of forces as described below, Fig. 5:

Let W = weight of strip (abcd).

C = apparent cohesion of soil.

ϕ = angle of shearing resistance.

L = length of arc of the strip.

θ = angle of inclination of the tangent to the horizontal.

Now, resolving the weight W tangentially and normally to the arc,

$$T = \text{shear force} = W \sin\theta$$

$$N = \text{normal force} = W \cos\theta$$

$$\therefore \text{ shearing resistance} = CL + W \cos\theta \quad \tan\phi$$

Hence, for the entire section, the factor of safety against slipping can be expressed as :

$$F.\ S. = \frac{\text{Shearing resistance}}{\text{Shearing force}} = \frac{\Sigma\ (CL + W \cos\theta\ \tan\phi)}{\Sigma\ W \sin\theta} \quad ...(10)$$

3.5.10. From the stability analysis, the strength required for the soil to be stabilized is compared with the results of the unconfined compression test of soil samples. Sowers and Sowers have given *rough* indication for use of factor of safety for different purposes as shown in Table 3.

TABLE 3

Factor of safety	Purpose on which used
Less than 1	Unsafe
1.0 to 1.2	Questionable
1.3 to 1.4	Satisfactory for cut, fill, etc., but questionable for dam
1.5	Safe for dam

4. LANDSLIDES ON DIMAPUR-IMPHAL ROAD AND SOIL TYPE STUDIES

4.1. Description of the Road

The Dimapur-Imphal road, a part of National Highway 39, is the main artery and lifeline of communication connecting Nagal Hills district and Manipur State with the plains districts of Assam Plate I. Before the railway line was established, the road was meant for cart traffic to transport commodities and carry local business with other parts of Assam, and for this reason the old name of the road was "Dimapur-Imphal cart road". The opening of Assam—Bengal rail-road rendered

Dimapur, known as Manipur Road railway station, an important place from which this road starts. The road runs in a more or less south-easterly direction with a wide convex curve towards north-east and the distance is about 134 miles up to Imphal.

4.2. Geology of the Area

4.2.1. The geologists believe that the natural tearing movements of the Himalayas are still active since millions of years and extend up to the sub-Himalayan belts. The structural analysis of rocks indicates that such pressure tends towards the hilly regions of Assam Valley mainly around North Lakhimpur area. The types of rock formations in all the hilly areas of Assam are gneiss, sandstone, limestone, trap, granite, Khasi greenstone, Amphibolite and Pyroxeme Gramilite, Dihing, Barail, Kopili, Disang, and Shillong series as shown in Plate II.

4.2.2. In 1882, R. D. Oldham, a geologist, first traversed the whole Naga Hills district and he found two principal rock formations, viz., upper tertiary and axials on east and west of Kohima. He also suspected the fault line in between those two series near about Kohima town. After Oldham, tertiary formations were reclassified by the geologists into two series, viz., Barail and Disang series. Barail series is composed of arenaceous beds of sandstones with greater thickness. It derives its name from the Barail range in north Cachar Hills which emerges from Jaintiapur, crosses through North Cachar and continues up to Kohima in Naga Hills. The area through which this series passes is hilly with rugged topography. All the high hills ranging from Kohima including Japdow, the highest peak in Assam (9,890 ft above M.S.L.), belong to Barail series built with massive sandstones and sandstones mixed with shales.

4.2.3. Disang series which is older than Barail is composed mostly of shales directly underlying the Barail series and is more argillaceous and contorted than massive sandstones of Barail series. In the Kohima area, the junction of these two series is faulted as may be seen from the map prepared by Oldham, Fig. 6. The Dimapur-Imphal road passes very closely to this faulted zone. According to geologists, a strong thrust, crushing and crumbling, exists in the faulted area which allows the Barail series to push the Disang series in north-easternly direction. Thus, the whole area is susceptible to severe landslides and subsidences.

4.2.4. While travelling along this road, it will be conspicuous that extensive slips occurred long ago, Fig. 7. The depressions where wet cultivation is done in man-made terraces are nothing

Photo 1.
Damaged retaining walls due to landslides

Photo 2.
Heavy landslides in Mishmi Hills due to earthquake of 15th August 1950

Photo 3.
Installation of horizontal drains to decrease the hydrostatic pressure

Photo 4.
Hill faces sliced down due to earthquake of 5th August 1950 in Abor Hills

Photo 5.
Rock falls and bedding of sand stones

Photo 6.
Landslides due to flowage

Photo 7.
Landslides in bare hills due to disintegration and weathering

Photo 8.
Horizontal drains working to release the hydrostatic pressure

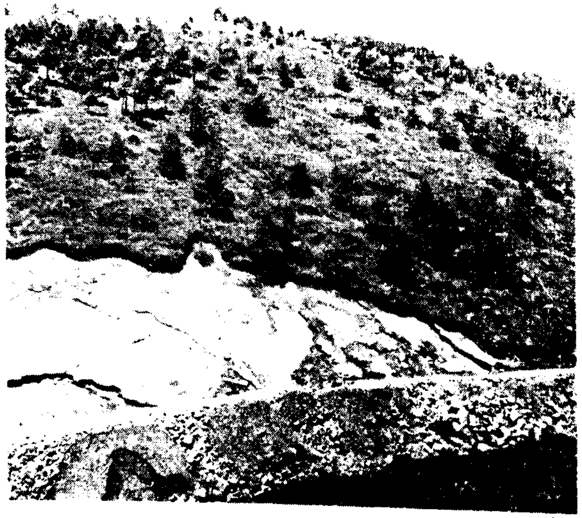


Photo 9.

Face of secondary slips

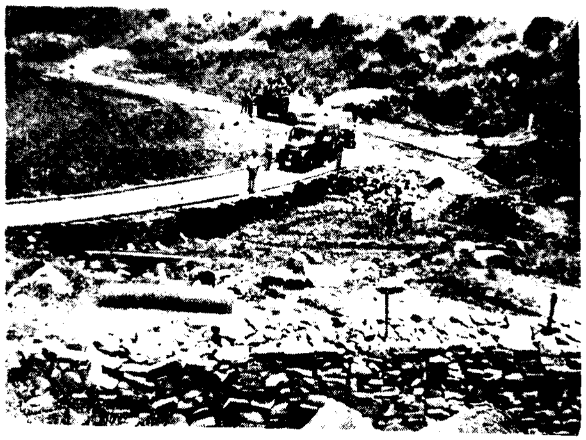


Photo 10.

A diversion with temporary bamboo bridge due to slumping of road

Photo 11.

Disturbed hills due to earthquake of 1950 in Mishmi Hills

Photo 12.

Bare hills, boulder sausages and a new diversion opened for traffic

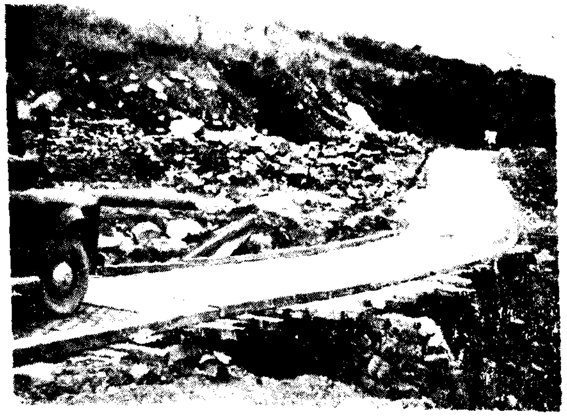


Photo 13.

Bridging to allow slip flowage

Photo 14.

Boulder sausage and jungle wood palisading

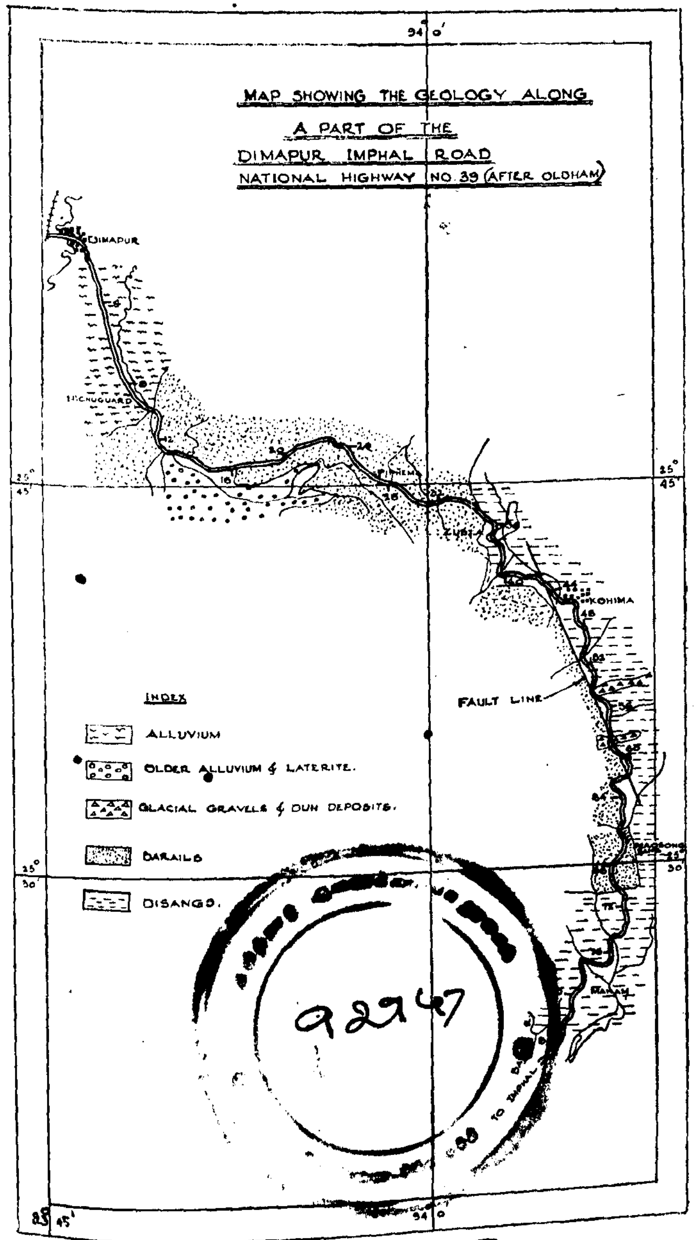


Fig. 6

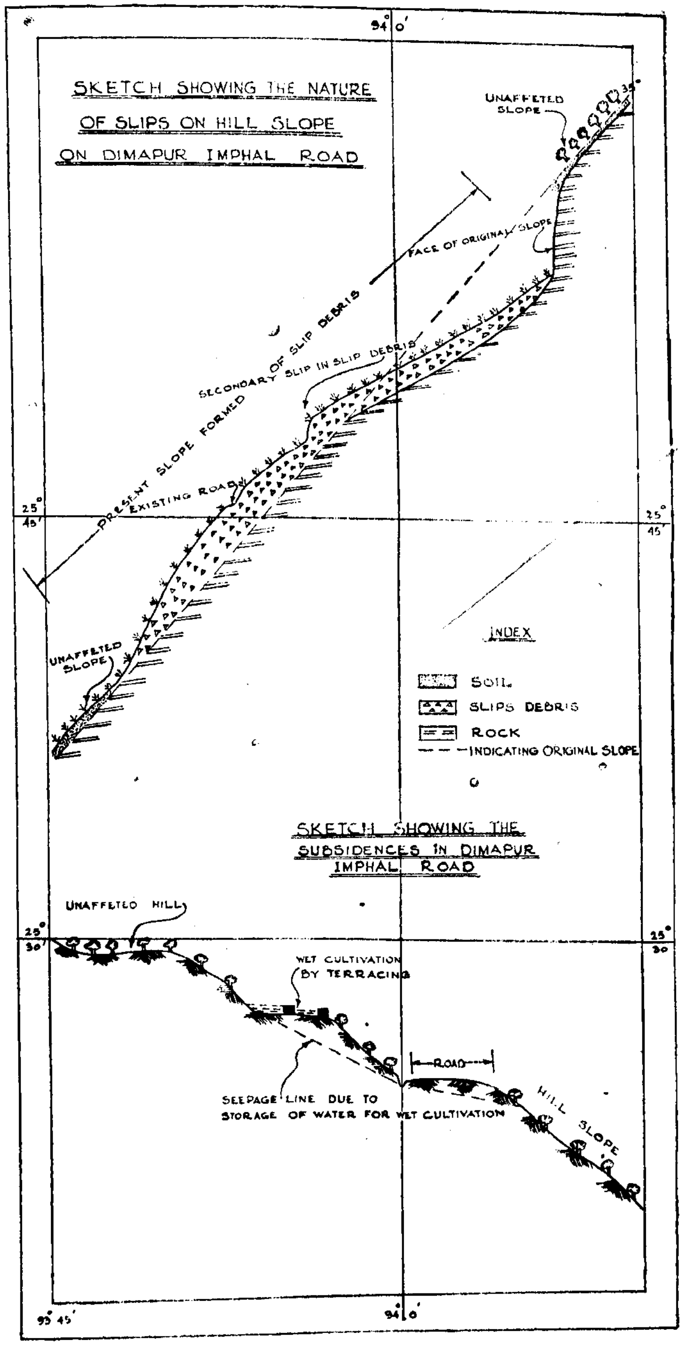


Fig. 7

but old landslides. Some deposits composed of sub-angular boulders mixed with debris of varying degrees of fineness may also be seen which were described by Major Godwin Austin as glacier moraines in 1872 when he traversed Naga Hills district in connection with the topographical survey. This was of course subsequently contradicted by Oldham who stated that these deposits were due to heavy disintegration of rocks aggravated by excessive rains and decrease of forests in slopes from rapid variation of temperature.

4.3. Meteorology

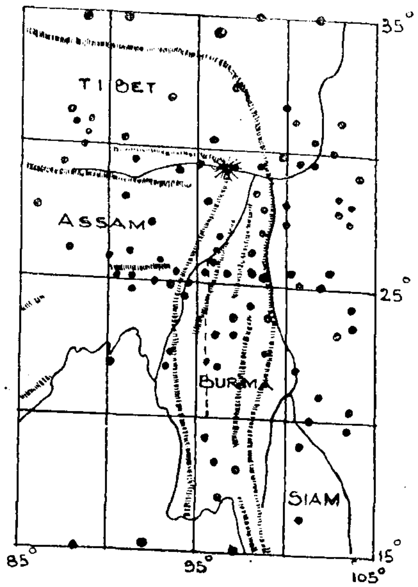
4.3.1. Throughout the year, hillsides are not disturbed except during the period of monsoon which generally starts by the later part of March and continues up to the beginning of October. The heaviest precipitation usually occurs during the months of June to August with an average yearly rainfall of 75 inches. The yearly mean relative humidity is approximately 88, mean maximum temperature 70°F., mean minimum temperature 58°F., the highest maximum temperature 90°F. in April and the lowest minimum temperature 41°F. in January at Kohima, as may be seen from Annexure 2. Annexure 3 shows the variation of rainfall, temperature, etc., in different months of different years. Also see Plates III and IV.

4.4. Seismology

4.4.1. Earthquakes are primarily responsible for many landslides both small, very large and disastrous ones. They produce horizontal accelerations which may increase or decrease the shearing stresses of soil mass. Vibrations due to blasting, machinery and heavy traffic may also produce transitory stresses. From the map in Fig. 8, it will appear how the areas where the Dimapur-Imphal road passes are affected by earthquakes. Due to seismological conditions, the area is quite susceptible to heavy landslides and it is suggested that in designing retaining structures, the maximum acceleration of 0.2g for rock foundations and 0.5g for alluvium may be adopted on account of earthquake shocks in Assam.

4.5. Mineralogy

4.5.1. To study the landslide problem, mineralogical study and petrology of soils and rocks are essential to know the physical and chemical properties of each mineral which may also help in dealing with preventive and control measures of landslides.



MAP SHOWING EPICENTRES OF AUGUST 15, 1950. AND PREVIOUS YEAR, AXES OF MOUNTAIN RANGES NAGATIVE GRAVITY ANOMALIES.

INDEX :-

EPICENTRE FOR AUGUST 15TH EARTHQUAKE

PREVIOUS EPICENTRES.

HATCHED LINES — AXES OF MOUNTAIN RANGES.

BROKEN LINES — NAGATIVE GRAVITY ANOMALIES.

Fig. 8

The following tests may be carried out in the laboratory :

(1) **X-ray Diffraction** : This test will help to ascertain the diffraction pattern of unknown minerals and assess the reactions in analogy to other similar types of known minerals.

(2) **Thermal Analysis** : This helps to identify the minerals from their thermal properties.

(3) **Petrography** : This will assist to know the physical properties of mineral grains with the help of the microscope.

Due to want of testing facilities within the State, the tests mentioned above could not be carried out on rocks and soil minerals and were left for future studies.

4.6. Description of Landslides and Subsidences

4.6.1. The Dimapur-Imphal road is affected by landslides and subsidences mainly in the portion between Nichugard and Maram, i.e., from mile 9 to 82. As the magnitude of slips varies in every rainy season, only a general account of some of the landslides is furnished here. The nature of slips and their causes are almost identical in all cases as the rock formation of the whole area, where landslides occurred, is of the same type, viz., shattered Disang shales. These landslides have been very active since the last 25 or 30 years, but no detailed information is available to show the variation of movements and damages since their first appearance.

4.6.2. **34th mile** : In this area, a portion of the road about 600 ft in length has subsided due to creep. More than 200 ft length of the hills has moved downwards in the direction of the slope. In cuttings of hill sides, soil mixed with fragments of black shale is visible. The road bed consists of bedded shales and mud stones with variable dips. The slope above and below the road bed is undulated and fairly steep about 30 degrees or so. Due to seepage, this creeping movement takes place dragging the road away from the original position. The slide in this mile has been active since 1941. In 1946, the road was blocked for a period of two and a half months. The slide of the whole area has moved from the top of the hill as a single block. In 1956, there was a subsidence of about 100 ft in length. The rain water was the main destructive agent activating the heavy landslides in this portion. During rains, the loose earth becomes muddy and moves downward at a speed of 20 ft to 25 ft per hour.

4.6.3. **40th mile** : A length of the road of about a furlong, in mile 40, is badly damaged and subsided. The sliced slope of the hill indicates black shales mixed with soil. The road bed is located on the debris of old slides. There is little vegetation in the hills in this slipping zone and the slope of the hill varies from 30 to 35 degrees all along.

4.6.4. **41st mile**: The slip in mile 41 was active since 1941 and severe in 1947 and 1950 when the road was blocked for about one and 3½ months respectively, and a length of 600 ft of the road in furlong 4 was badly damaged during 1950. The road cutting shows pieces of deep coloured shales mixed with fine grained soil. The road is aligned over the debris of old slides which may be surmised from the barenness of hills without vegetation and valley like depressions here and there. Wet cultivation is done in terraces on the steep hill slopes. The landslides have taken place mainly due to seepage of water through porous soils and creeping movements along the slipping plane.

4.6.5. **42nd mile**: During 1950, the 5th furlong of 42nd mile was completely blocked for more than 3 months due to heavy landslides, which were active since 1941. The road about 250 ft in length slumped in 1950. The slope of the hill is 30 degrees or more and the soil is mixed with clay and shales. There are indications of debris of old slides over which the road has been aligned. Temporary diversions were constructed for traffic movement which also failed on account of landslides in subsequent rainy seasons.

4.6.6. The other portions of the road where major slips have occurred are in miles 12 and 13, 23, 24 and 27, 46, 51 and 52, 64, 66, 68, 71 and 72, 74 to 76, and 79.

4.6.7. Maram is situated in mile 82. Beyond Maram, the road follows the top of spur of the hills. The cutting faces are shaly and fairly covered up with soil. Towards Imphal from Karing, the road is aligned through Barak Valley and no major landslides have taken place so far up to Imphal except small slips here and there in miles 85 to 90, 92 to 94, 102 to 104, 106, 107, 112, 113, 115, 124 and 125.

4.7. Soil Type Studies

4.7.1. Soil samples in greater number from all the sections in all the slip areas could not be collected and tested for lack of testing facilities available within the State. So, only a few representative samples from 34th, 40th, 41st and 42nd mile slips, where the landslides were more active and severe for a number of years, were collected and sent to West Bengal for test. The test results as obtained are furnished in the following Tables.

TABLE 4

Location of soil samples	Liquid limit per cent	Plastic limit per cent	Plasticity index per cent	Specific gravity
34th mile slip	34.4	18.8	15.6	2.68
40th ,, ,,	36.6	19.2	17.4	2.69
41st ,, ,,	38.4	17.7	20.7	2.66
42nd ,, ,,	43.6	19.7	23.9	2.71

TABLE 5

Location of soil samples	Moisture content per cent	Dry density in lb per cu. ft	Angle of internal friction in deg. ϕ	Cohesive strength of the soil in lb per sq. ft.	Unconfined compressive strength lb per sq. ft.	Capillary rise of water
34th mile slip	10.1	135.1	15° -31′	1692	4450	18 ft 2 in. @ 29.5°C.
40th ,, ,,	11.2	129.2	25° -28′	1757	5926	4 ft 6 in. @ 29.5°C.
41st ,, ,,	11.1	130.8	21°−34′	1433	4478	19 ft— 3 in. @ 29.5°C.
42nd ,, ,,	11.0	130.5	25°−20′	1512	4738	24 ft−10 in. @ 30°C.

TABLE 6

Location of soil samples	Moisture content at the point of saturation per cent	Dry density in lb per cu. ft.	Angle of internal friction in deg. ϕ	Cohesive strength in lb per sq. ft.
34th mile slip	14.5	120.0	7° 46′	98
40th ,, ,,	14.0	120.8	9° 30′	149
41st ,, ,,	14.5	122.4	16° 10′	201
42nd ,, ,,	14.0	122.1	17° 22′	454

4.7.2. The stability analyses are made with the test results of soil samples as shown in Tables 7 and 8. A specimen of detailed calculations both for soils in-situ and saturated soils is given in Annexures 4 and 5. Also see Annexure 6 for a typical worked out example.

TABLE 7

COMPARISON OF STABILITY OF SLOPES FOR SOILS IN-SITU AND SOILS SOAKED IN WATER

Situation	Factor of safety (soils in-situ)	Factor of safety (soils saturated in water)
34th mile slip	1.97	0.33
40th ,, ,,	2.17	0.49
41st ,, ,,	2.05	0.82
42nd ,, ,,	1.92	0.98

TABLE 8

COMPARISON OF STABILITY OF SLOPES WITH UNCONFINED COMPRESSION STRENGTH OF SOILS IN-SITU

Situation	Factor of safety (soils in-situ)	Factor of safety worked out from compressive strength of soils in-situ
34th mile slip	1.97	1.78
40th ,, ,,	2.17	2.02
41st ,, ,,	2.05	1.79
42nd ,, ,,	1.92	1.80

4.7.3. The factors of safety worked out from the test results of undisturbed soil samples, as seen from Table 7, are much higher than unity. From the unconfined compression test results also it is seen (Table 8) that the factors of safety of soils are more than unity. This shows that in normal conditions, specially in dry periods, the soils are quite stable. Laboratory tests show that when they are saturated during rains, the cohesive strength as well as the internal friction are considerably decreased while, on the other hand, the hydrostatic pressure is increased. Thereby, the slopes become unstable and sliding takes place along the critical surfaces.

5. ANALYSIS OF CAUSES FROM DATA COLLECTED AND METHODS OF TREATMENT

5.1. Analysis of Causes

5.1.1. As already explained in the previous pages, no slide is attributed to a single cause which can be definitely determined. Most of the landslides take place under the influence of more than one factor, viz., geological, topographical, climatic, physical and chemical conditions. The process of developing landslides starts from the beginning of the formation of rocks and continues with physical changes due to wind, water, erosion and weathering. From Fig. 6 it will be seen that the Dimapur-Imphal road is aligned along the fault line in between Barail and Disang series of rocks where great geological thrust is exerted between them. This area is always susceptible to landslides which cannot be altogether avoided by adopting any preventive or control measures whatsoever.

5.1.2. As seen from Fig. 8, the seismological turmoil in the area is still active which increases the stresses and reduces the shearing strength of soils or rock masses. It is thus quite natural to surmise that these natural forces are aggravating the landslides in this region.

5.1.3. Some of the dark coloured shales have deliquescent salts such as sodium chloride, sodium sulphate and potassium sulphate. These salts absorb water and moisture and have properties of breaking the shales into clay. During rains, these salts are readily dissolved and the mass, lubricated with clay, flows like bituminous material.

5.1.4. In 64th mile slips, tension cracks are quite prominent. During heavy rains in monsoon, water enters through these cracks and cavities and helps the whole mass to slide along the critical plane. These cracks are due to internal stresses of soil activated by the hydrostatic pressure and also the pressure of the slice of soil mass within the boundary of tension cracks and critical surface.

5.1.5. In this area, two types of creeping movements are experienced. In one case, the slips take place quietly due to dislodgment of a big area from the root to the tongue of the sliding mass. Even big trees are seen taking new position upright lower down the hills. The second case is confined to the loose materials with regular rolling. Such slides continue throughout the year and become more active during rains. The main reason for the first case is the breaking up of the strata of Disang series to the proximity of the critical surface in the faulted zone. The second case is due to soaking of surface water by soft materials during rains and converting them into semi-fluids which flow like lava in a volcanic action.

5.1.6. In many places, it is seen that the jungles are cleared and trees are removed by the villagers for jhoom cultivation and thereby the natural hill slopes including the top stable layer of the soil are disturbed. The deforestation causes the maximum runoff during rains and exposes the rock surfaces to rapid weathering. During the monsoon months, wet cultivation is done in terraces by impounding water up the steep hill slopes. Seepage of water through pervious strata takes place along the slipping plane and causes the landslides. Sometimes those impounding reservoirs issue numerous springs which wash away the hill face materials unscrupulously and make the hills much steeper than the angle of repose.

5.1.7. Some perennial streams are flowing through deep gorges inducing scours by the rush of water during rains. The bottom of the steep hill slopes is undermined on both sides of the stream resulting in landslides due to removal of back pressure.

5.1.8. In many places, the road is aligned over the debris of old slides which can be judged from the bareness of hills, less growth of vegetation and valley like depressions. In such places, the grade of the hill slopes varies from 50 to 70 degrees against the natural slope of 30 to 35 degrees. All secondary slips occur in the old landslide areas.

5.1.9. The stability analysis shows that the cohesive strength of the soil is reduced due to hydrostatic or pore water pressure. When the soil is fully saturated, it becomes much less so than what is actually necessary to have the required factor of safety. The ground water table rises with the increase of rainfall during monsoon. Along with it, the capillary tension of the soil is increased making the whole mass completely saturated and liquefied.

5.1.10. The landslides in this area became more intensified since the road was widened in 1941. By doing so, the natural slope of the hill was disturbed and shearing strength of the soil was reduced. In some places, new diversions were constructed

for immediate movement of traffic which also disturbed the natural stability of the hills.

5.1.11. Provision of unsystematic and inferior types of drains in the slip area without proper hydrological survey encourages rather than controls landslides. The proper design of such structures should generally be based on the correct hydrological data and true conditions of soil.

5.1.12. From Plate III, it is seen that the severe landslides occurred after the heaviest rainfall almost every year. During winter, the soil remains dry and as soon as monsoon starts, the rain water softens the soil mass, percolates through pervious strata, and induces the unstable mass to move along the plane of rupture. In many cases rain water lubricates the top soil strata and helps the slide due to flowage. Thus, the rain is the potential agent in this arid region to encourage landslides and subsidences.

5.2. Methods of Treatment

5.2.1. Prior to 1941, the importance of Dimapur-Imphal road was not much due to less traffic on the road and, therefore, much care for the treatment of landslides was not taken. Vagaries of slides were not so intensive prior to the widening of the road. Generally, the road remained closed for vehicular traffic till the spoils were removed by manual labour. Dry stone masonry retaining and breast walls were constructed to retain the debris from blocking the road and retard the forward movement of the sliding mass. For treatment of creeping movement, kutcha catchwater drains were installed to drain off surface water.

5.2.2. In subsequent years, i.e., during 1942, 1946, 1950, 1954 and 1956, the landslides in the portion between miles 9 and 82 were severe and heavy. During 1956, extensive works of surface drainage, construction of retaining and breast walls (Plates V and VI), and boulder sausages (Plate VII) were taken up. The kutcha catchwater drains were made watertight with stone masonry and a network of drainage system spread as per topography of the sliding areas. Slopes were flattened, benching of slopes with boulder sausages at toes was taken up as stable foundations for retaining structures were not available within a reasonable depth. Buttresses, breast and retaining walls were constructed wherever possible to retard the creeping movement. See Annexures 7, 8 and 9 for typical worked out examples of the various structures.

5.2.3. Where landslides occurred due to flowage, jungle wood spurs were constructed in series at intervals along the slope of the hills which retained the surface flowage. By doing so, the debris of slides were retained from moving downwards till they obtained the natural angle of repose and were stabilized along with the growth of vegetation. Where the slides took place due to erosion of river in gorges, series of boulder sausages were made which raised the river bed and stopped landslides by backwater pressure. The above measures gave good results and there were no severe landslides during 1957 and the current year till now.

5.2.4. In general, no standard of corrective measures to be adopted in dealing with landslides can be specified as each landslide requires special investigation and different treatment, as already stated in the previous pages. The following are the possible methods so far advanced by which landslides are eliminated or controlled. After theoretical, physical and laboratory analyses, one or more of the following measures suitable to each particular problem may be tried:

(1) Relocation of the road by completely abandoning the slip area with comparative study of cost of such relocations and protective measures proposed to be resorted to.

(2) Removal of landslides completely by cutting and dredging up to the bed rock in the slipping plane. This can be ascertained by taking a series of borings to the maximum depth and then by plotting them into the log.

(3) Bridging the slip area if it is small and two stable ends are available.

(4) Construction of cribbing of either concrete or timber.

(5) Cementation of loose materials in the entire area by any other method such as cement grouting, bitumen grouting, etc.

(6) Retaining walls, breast walls either of masonry or concrete or boulder sausages.

(7) Piling with steel, concrete or timber.

(8) Network of surface drainage system in the entire slip area.

(9) Slope treatment.

(10) Sub-surface drainage of French type.

(11) Vertical and horizontal sand drains.

(12) Chemical treatment. Clayey, waterlogged and sandy soils may be hardened by spraying a strong solution of calcium or magnesium chloride.

(13) Tunnelling.

(14) Sealing the joints in open fissures, peripheral cracks, etc.

(15) Buttresses.

(16) Steel dowels grouted into bed on top of which sliding occurred and anchored with concrete blocks or light walls near the upper edge of the sliding areas.

(17) Perforated horizontal drains to release the hydrostatic pressure.

(18) Method of soil stabilization by injecting asphaltic emulsions, cement grouting, electro-osmosis or electro-chemical treatment.

(19) Stabilization by re-seeding with green deep-rooted vegetation or ecological treatment.

(20) Drainage by tunnelling.

(21) Underground walls by filling drill holes with cement grouting.

(22) Sluicing. The stream is controlled from top and directed where washing off the detritus materials to get the hillside to natural slope is necessary.

5.2.5. In selecting an alignment, specially in locating cutting, ecological observations should be made regarding appropriate planting since special types of plants characterise the area and drainage while doing the preliminary survey of the alignment. Exposed surfaces should be covered up with top soil and suitable types of plants allowed to grow. The surface erosion may be minimised by fixing wooden poles diagonally across the face of the cutting to divert the flow of surface water and check the downward movement of soil particles.

5.2.6. Without going into much detail when facilities are not available for such investigations, the following methods may be tried as control measures according to the classification of landslides mentioned in previous pages:

(1) Natural landslides in homogeneous soil: piling, proper drainage of surface water, growth of proper vegetation and underdrainage.

(2) Natural landslides in heterogeneous soil: surface drainage, underdrainage by tiles, well drainage, tunnelling, piling and grouting.

(3) Slide due to plastic flowage:
homogeneous clay: piling and retaining walls.
Sandy soil: grouting and piling.
Creep: retaining walls and surface drainage.

(4) Overhanging strata and collapse of soil mass:
Cohesive soil: retaining walls and piles.
Pervious soil strata: reverse filling and grouting.

6. SUMMARY AND RECOMMENDATIONS

6.1. The following recommendations are particularly made for prevention and control of landslides on Dimapur-Imphal road. The implementation depends on the economic analysis of each type of operation according to the needs and resources available.

(1) As the road was aligned in a geologically unsound area, the permanent solution to get rid of landslide occurrences will be to divert the road to a safer zone away from the proximity of faulted line. If such overall deflection is not feasible, at least diversions in slip areas should be constructed away from the fault line and founded on the rocks of Barail series instead of Disang series.

(2) Indiscriminate deforestation for any purpose should totally be avoided. Shifting cultivation or jhoom cultivation should be restricted in steep hills, specially in areas susceptible to landslides.

(3) In case of widening the road, steep hill cuttings should be eliminated and proper angle of repose for different types of soils and rocks should be allowed and strictly followed.

(4) All unstable materials above the plane of rupture should be totally removed if the depth is shallow and economically possible.

(5) Benching of slopes as per theoretical analysis should be adopted to decrease the shearing stresses of soil.

(6) Construction of diversions over the debris of old slides should be avoided. Such locations can be traced out from the bareness of hills, depressions in slopes, examination of soils after boring for constituents of fragments of shales, sandstones, and fine grained clay.

(7) The slope may be stabilized by spraying heavy grade oil to seal the water passages through pores to decrease the capillary tensions and seepages. Loose soil may also be hardened by spraying at first a strong solution of calcium chloride or magnesium chloride and then a strong solution of sodium hydroxide or potassium hydroxide during dry period.

(8) Stability analysis of landslides on the following formula explained in the previous pages gives satisfactory result provided undisturbed soil samples are properly tested and correct results obtained:

$$T = N \tan \phi + CL.$$

(9) In case of toe cutting in gorges by perennial streams, boulder sausages constructed in series raise the river beds and reduce the landslides due to backwater pressure.

(10) Boulder sausages should be constructed at the toe of hill slopes where landslides occur to keep away the loose dry masses of soil and rock falls from the road surfaces when solid foundation is not available for other types of restraining structures such as retaining walls, buttresses, breast walls, etc.

(11) When it is seen from the theoretical analysis that the shearing strength of the soil is reduced, restraining structures like buttresses, retaining walls, boulder sausages, jungle wood spurs, etc., should be constructed to counterbalance it. Piling will not give satisfactory result due to creeping and flowage. Series of jungle wood spurs may be constructed in hill slopes to retain the loose debris till they obtain the angle of repose. To stabilize them further, deep-rooted grass may be planted.

(12) The water in this arid region is a potential agent which reduces the shearing strength and increases the shearing stresses of soils. So, a thorough analysis of its action and appropriate remedies are equally important. The following suggestions will render beneficial effects on correct implementation;

(a) **Surface drainage:** Hydrological and soil surveys should be carried out and a network of surface drainage system planned on present topography of the slided areas. Properly designed drainage structures should be watertight to prevent percolation through porous soil. Adit drains on steep hill slopes and catchwater pits with easy outlets will help to drain off the surface water. Dropping of water through culverts in downhill sides should be prevented by providing pucca steps to stop side erosion and downhill landslides.

(b) **Cracks and cavities:** Cracks and cavities in slopes should be sealed to prevent water entering into them during rains which helps the creeping movement of the sliding mass along the slipping plane or plane of rupture.

(c) **Fissures**: When fissures of rocks are prominent, they should be grouted in cement to stop weathering and erosion by wind and water. During rains, water enters into the rocks through these fissures, softens the cementing materials and helps them fall down due to gravitational forces.

(d) **Artificial reservoirs:** Man-made impounding reservoirs in depressions of steep hill slopes issue numerous small streams which wash away the stable materials from the surface and expose the rocks to rapid weathering.

(e) **Slope treatment:** Treatment of slopes is essential for easy drainage of run-off water during rains. Grades should be flattened, and turfing and afforestation taken up expeditiously. Ecological treatment of slopes is a vital point to deal with landslides.

(f) **Sub-surface drainage**: The water table line should be lowered down specially during rains to decrease the hydrostatic pressure and seepage of water which can be achieved by installing horizontal and vertical drains of any type suitable to the local conditions as described in the previous pages.

6.2. A complete and thorough search should be made to ascertain the actual causes of landslides and appropriate measures taken accordingly with a view to reap out the full benefit of the investments made.

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Annexure 1

Statement showing the Expenditure Incurred in dealing with some Landslides on Dimapur-Imphal Road

Landslides situation in miles	EXPENDITURE										Total expenditure for last 5 yrs. both repaid & original against each mile slip
	1953-54	1954-55	1955-56	1956-57	1957-58	1953-54	1954-55	1955-56	1956-57	1957-58	
34	18358	13300	8140	1300	1400	Nil	3900	Nil	3250	6800	56448
40	1400	1200	8240	1100	1300	Nil	3900	Nil	3927	16500	37567
41	4200	1300	8040	1300	1400	Nil	3900	Nil	17700	29000	66840
42	4200	1200	8140	1200	1300	Nil	3900	Nil	31700	70000	121640

Annexure 2

Rainfall and Temperature at Kohima

Year	Total rainfall during the year in inches	Yearly mean relative humidity in per cent		Yearly mean maximum temperature in °F	Yearly mean minimum temperature in °F	Highest maximum temperature during the year in °F	Lowest minimum temperature during the year in °F
		8.30 hrs.	17.30 hrs.				
1955	70.05	81	88	69.9	58.7	84° on 29-6-55	41° on 1-1-55
1956	75.11	77	81	69.9	58.5	90° on 19-4-56	41° on 31-1-56
1957	57.91	72	79	70.0	58.1	89° on 21-5-57	42° on 14-1-57 & 15-2-57

Annexure 3

Rainfall, Temperature and Humidity at Kohima

Year	1954					1955					1956					1957				
	Rainfall	Temperature		Humidity		Rainfall	Temperature		Humidity		Rainfall	Temperature		Humidity		Rainfall	Temperature		Humidity	
		Max.	Min.	8.30 hrs.	17.30 hrs.		Max.	Min.	8.30 hrs.	17.0 hrs.		Max.	Min.	8.30 hrs.	17.0 hrs.		Max.	Min.	8.30 hrs.	17.0 hrs.
January	0.43	57.6	45.7	55	82	0.48	59.8	46.5	90	99	1.05	56.3	45.1	62	80	—	14.1	7.6	82	86
February	1.99	64.9	52.1	64	78	0.19	62.7	49.8	90	97	1.10	62.6	49.6	57	66	0.38	15.3	8.1	70	71
March	1.69	77.3	62.9	65	64	2.16	72.0	57.2	76	94	3.09	70.4	56.2	59	61	17.00	20.2	12.2	65	53
April	1.40	71.8	58.4	49	59	4.03	73.5	59.8	64	55	7.56	76.8	60.4	59	65	39.10	24.9	16.8	48	54
May	5.64	77.3	65.1	76	81	6.75	74.9	64.5	80	86	21.44	75.2	64.3	87	83	1.18	26.3	18.1	59	62
June	16.13	74.9	66.1	89	93	10.45	75.4	65.6	90	91	11.14	74.3	62.0	89	90	12.26	24.6	18.7	78	[illegible]
July	9.30	74.9	66.4	91	96	11.48	74.3	65.5	89	95	15.47	75.0	65.0	90	90	12.92	24.1	19.1	89	94
August	9.20	75.2	66.5	90	95	15.68	74.3	64.9	90	96	15.02	75.8	66.1	90	91	13.89	24.7	19.1	88	94
September	11.54	76.2	66.0	83	91	7.49	75.2	65.3	81	90	6.45	75.4	65.4	81	84	10.01	24.0	18.5	82	90
October	11.30	69.7	59.9	86	96	4.63	72.5	62.5	80	90	3.61	71.9	61.4	83	87	3.20	21.2	15.2	72	87
November	0.32	65.1	52.9	88	98	6.66	66.9	56.0	78	83	2.53	65.4	54.9	81	90	3.00	18.8	11.8	63	85
December	0.22	60.5	48.2	93	97	0.32	58.4	47.0	63	83	0.35	59.5	47.6	81	86	0.30	55.5	47.0	73	84

Annexure 4

Stability analysis in mile 34

(Soil as per Table No. 5)

	0–1	1–2	2–3	3–4	4–5	5–6	6–7	7–8	8–9	9–10
Increment area (sq. ft.)	81.4	210	212.5	395	4425	4575	455	425	360	210.1
Increment weight (Kips) (Unit wt. 135.1 lb per cu. ft.)	11.0	28.4	28.7	53.5	60.0	62.0	61.5	57.5	48.6	28.4
Tangential force (Kips)	7.1	14.0	9.45	8.75	4.06	13.7	24.2	32.4	35.2	25.7
Normal force (Kips)	8.4	14.5	27.3	52.5	59.6	61.5	57.2	47.2	32.9	13.5

$\therefore \quad \Sigma T = 174{,}600$ lbs $\qquad \phi = 15° - 31'$

$\Sigma N = 384{,}600$ lbs $\qquad C = 1692$ lb per sq. ft.

$L = 140$ ft

$$\therefore \quad F.S. = \frac{\Sigma N \tan \phi + CL}{\Sigma T} = 1.97$$

Annexure 5

Stability analysis in mile 34

(Soils in saturated condition as per Table No. 6)

Unit weight = 120 lb per cu. ft.

	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10
T (Kips) ...	6.3	12.5	8.45	7.8	3.6	12.1	21.5	28.7	31.1	22.8
N (Kips) ...	7.45	21.7	24.4	46.7	52.8	54.5	50.6	41.8	29.1	12.0

$\therefore \ \Sigma\, T = 154{,}900$ lb $\qquad \phi = 7^\circ - 46'$

$\Sigma\, N = 341{,}100$ lb $\qquad C = 98$ lb per sq. ft.

$L = 140$ ft

$$\therefore \quad F.S. = \frac{\Sigma\, N \tan \phi + CL}{\Sigma\, T} = 0.33.$$

Annexure 6

Method of Stability Analysis

The following is an example of typical computations in the stability analysis of landslides as described in the main study. One of the cross sections of the slip in 42nd mile of Dimapur-Imphal road has been taken for illustration.

The cross section is drawn to a scale and then the critical surface or the slipping plane is shown from theoretical calculations as described in the previous pages. Due to boring difficulties, the actual critical surface could not be ascertained from the strata of soils.

After drawing the slip surface, the slide area is divided into eight equal increments. The area of each increment is computed with the help of the planimeter and multiplied by the unit weight of the soil in-situ. The total weight of all the increments is obtained in a similar manner considering each increment in a width of one foot.

The weight of each increment is graphically resolved into tangential and normal components as shown in Fig. 9.

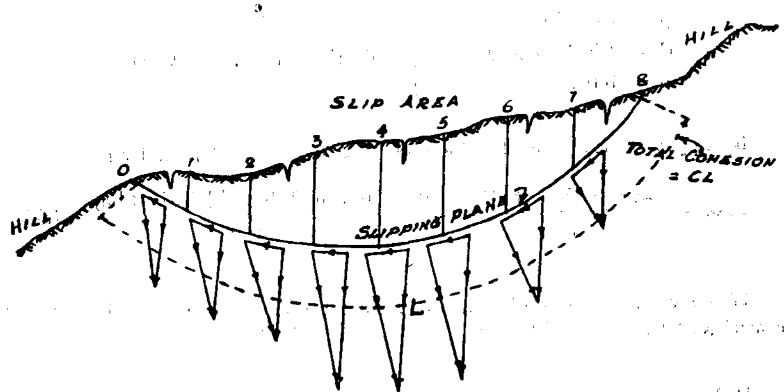

Fig. 9

VALUES OF TANGENTIAL AND NORMAL FORCES IN THE STABILITY ANALYSIS

Increments

	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8
Increment area (sq. ft.)	77	180	245	275	280	270	230	130
Increment weight (Kips) Unit wt=130.5 lb/cu. ft.	10	23.5	32	35.9	36.6	35.3	30	17
Tangential force (Kips)	6.05	9.2	6.5	1.6	8.9	16.5	20.6	14.6
Normal force (Kips)	6.85	21.5	31.8	35.7	35.6	31	22	8.5

$\Sigma T = 83{,}950$ lbs

$\Sigma N = 192{,}950$ lbs

$\left.\begin{array}{l}\phi = 25^\circ-20' \\ C = 1{,}512 \text{ lb/sq. ft.} \\ L = 100 \text{ ft}\end{array}\right\}$ obtained from soil test)

By applying formula (4), the factor of safety is obtained as:

$$F.S. = \frac{\Sigma N \tan\phi + CL}{\Sigma T} = \frac{192{,}950 \tan 25^\circ-20' + 1{,}512\times100}{83{,}950} = 1.92.$$

When values of C and ϕ are not known, one of them may be assumed to work out the value of the other unknown variable.

In the above example, the factor of safety comes to be greater than unity and may be considered as safe. But in some cases, such factor of safety becomes exorbitantly high which may be due to computations based on test data of soil samples collected after the landslides had occurred. Irregularities in the soil, difficulties in obtaining undisturbed soil samples in perfect condition, problems of laboratory test technique and effect of

hydrostatic and seepage forces are also some of the contributing factors that lead to such a high value of safety factors.

In case the water table line is above the slipping plane, formula (6) may be used in the same manner as described above to take care of the intergranular forces. The shearing resistance thus computed will indicate the strength needed to maintain the equilibrium of forces prior to subsequent movements.

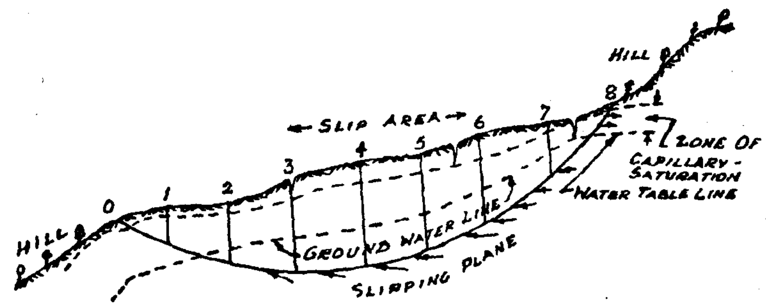

Fig. 10

Taking an arbitrarily chosen ground water table line as shown in Fig. 10, values of intergranular forces for different increments of the given cross section are computed as follows:

	0-1	1-2	2-3	3-4	4-5	5 6	6-7	7-8
Increment area (sq. ft.)	77	180	245	275	280	270	230	130
Increment weight (Kips)	10	23.5	32	35.9	36.6	35.3	30	17
Tangential force (Kips)	6.05	9.2	6.5	1.6	8.9	16.5	20.6	14.6
Normal force (Kips)	6.85	21.5	31.8	35.7	35.6	31	22	8.5
Intergranular force (Kips) $(hr_w l)$	5.2	9.3	11	12.5	13.5	12.5	11	8

$$\Sigma T = 83{,}950 \text{ lb} \qquad \phi = 25^\circ\text{-}20'$$

$$\Sigma N = 192{,}950 \text{ lb} \qquad C = 1{,}512 \text{ lb per sq. ft.}$$

$$\Sigma \mu = 83{,}000 \text{ lb} \qquad L = 100 \text{ ft}$$

$$\Sigma (N - \mu) = 109{,}950 \text{ lb}$$

$$F.S. = \frac{\Sigma (N - \mu) \tan \phi + CL}{\Sigma T.} = \frac{109{,}950 \tan 25^\circ - 20' + 1{,}512 \times 100}{83{,}950} = 1.4.$$

For expansive clayey soils, values of ϕ are found to lie within the range of 0 to 10 degrees. Hence, if the soil of the given cross section is considered to be clayey, the unknown variable C (cohesion of the soil) may be computed from the equation in formula (4)

Assume $\phi = 0$ degree,

$$\text{Then,} \quad C = \frac{83{,}950 - (192{,}950 \times 0)}{100} = 839.5 \text{ lb per ft}$$

Again, assume $\phi = 10$ degrees.

$$C = \frac{83{,}950 - (192{,}950 \times 0.1763)}{100} = 497.5 \text{ lb per ft}$$

In consideration of hydrostatic pressure with $\phi = 0$, the value of C will be the same as above.

Assuming $\phi = 10$ degrees, the value of C is obtained as :

$$C = \frac{83{,}950 - (109{,}950 \times 0.1763)}{100} = 645.6 \text{ lb per ft}$$

It is also possible to find out the cohesion required to give a true factor of safety of the same magnitude with respect of cohesion obtained from laboratory tests as shown below :

$$\phi D = \frac{\phi}{F.S.} = \frac{25^\circ\text{-}20'}{1.92} = 13^\circ\text{-}2'$$

$$\therefore \; C = \frac{(83{,}950 \times 1.92) - (193{,}000 \times \tan 13^\circ\text{-}2')}{100}$$

$$= \frac{161{,}000 - 45{,}300}{100} = 1157 \text{ lb/sq. ft.}$$

Annexure 7

Retaining Devices

1. PILING

In some cases, to increase the factor of safety, some external forces are necessary, which may be attained by driving piles penetrating the slipping plane or critical surface of sliding masses. These piles will produce additional resisting forces that will be added to the shearing resistance. The depth of penetration should not be less than one third of its length below the slip surface. The resistance to shear developed by the piles at the surface of rupture may be determined by applying the following formula advanced by Hennes :

$$S = \frac{Ap. \; fv}{D} \qquad \text{..............} \qquad (1)$$

where $S =$ shearing resistance of the pile in lb per foot.

$Ap =$ cross sectional area of the pile in sq. inch.

$fv =$ allowable shearing stress for the piles in lb per sq. inch.

$D =$ centre to centre spacing of piles in foot, divided by number of lines of pile.

Hennes has also suggested another formula as given below to check up whether the pile is resisting the shear of the soil along each side of the pile :

$$S = 2 \; Chd \qquad \text{..............} \qquad (2)$$

where $S =$ shearing resistance per pile in lb.

$C =$ cohesion of the soil in lb per sq. ft.

$h =$ height from slip surfaces to ground surface in ft.

$d =$ diameter of the pile in ft.

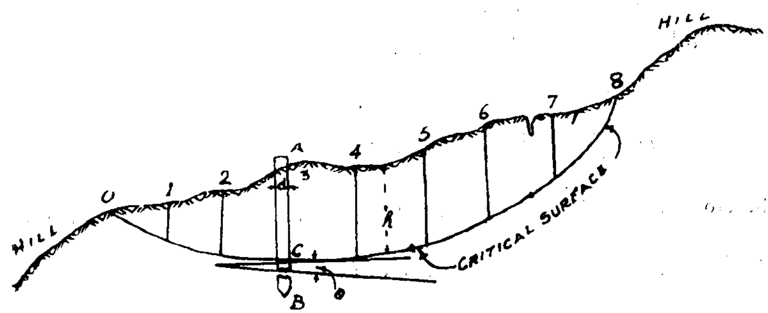

Fig. 11

Referring to the Fig. 11, let line 3 be selected for location of piling. Now resistance required from piling is given as:

$$P_R = F.S. \times \sum_3^8 T - \sum_3^8 N \tan\phi - CL \quad (3-8)$$

$\therefore$ Horizontal force, $H = P_R \cos\theta$

Assuming the portion BC of the pile to be fixed and taking triangular diagram of loading,

Cantilever moment $M = P_R \cos\theta \, D \frac{h}{3} \times 12$

where M = Maximum bending moment per ft width in inches.

h = Length of pile above slipping plane, AC.

D = Centre to centre spacing of piles in ft divided by number of lines of piles.

Then, $\sum_3^8 T = 62{,}200$ lb

$\sum_3^8 N = 132{,}800$ lb

$L = 75$ ft

Assuming a factor of safety=1.5, with the values of $\phi = 10°$ and $C = 497.5$ lb/sq. ft., $P_R = 1.5 \times 62{,}200 - 132{,}800 \times 0.1763 - 497.5 \times 75 = 32{,}500$ lb.

$\cos\theta = 0.923$

$h = 22$ ft (measured from the section)

Assuming that eight lines of piles are to be driven at 15 inches apart centre to centre, then

$$D = \frac{1.25}{8} = 0.156.$$

Therefore, $M = 32{,}500 \times 0.923 \times 0.156 \times 22 \times 12$

$= 410{,}000$ lb. in.

Selecting 12 inches diameter sal piles, we get

$$Ap = \pi \times 6^2 = 113.1 \text{ sq. inches.}$$

$$I = \frac{\pi}{4} \times 6^4 = 1017 \text{ ins}^4, \; Y = 6 \text{ inches.}$$

Therefore, $F = \frac{M.Y}{I} = 410{,}000 \times 6 = 2{,}100$ p.s.i. (may be allowed).

For shearing resistance of the pile, since $f = 200$ p.s.i.

$$S = \frac{113.1 \times 200}{0.156} = 145{,}000 \text{ lb per ft}$$

Therefore, $F.S. = \frac{S}{PR \; \text{Cos}} = \frac{145{,}000}{30{,}000} = 4.83.$

For stability with reference to the shearing of the soil,

$$S = \frac{2 \times 497.5 \times 22 \times 1}{0.156} = 141{,}000 \text{ lb per ft}$$

Therefore, $F.S. = \frac{141{,}000}{30{,}000} = 4.7.$

2. BUTTRESSES

For design of any retaining structure, it is necessary to ascertain the shearing resistance along the slip surface and also the resistance exercised by each retaining device adopted. Such a structure should be so located that the slipping plane may cross the base of the structure. The relative safety factor should be at least 1.5 for such structures.

Buttresses are often used as a retaining device to prevent landslides and creeping movements in hill roads. Failure of such structures takes place due to shear through them, foundation failure and shear between the structure and the foundation.

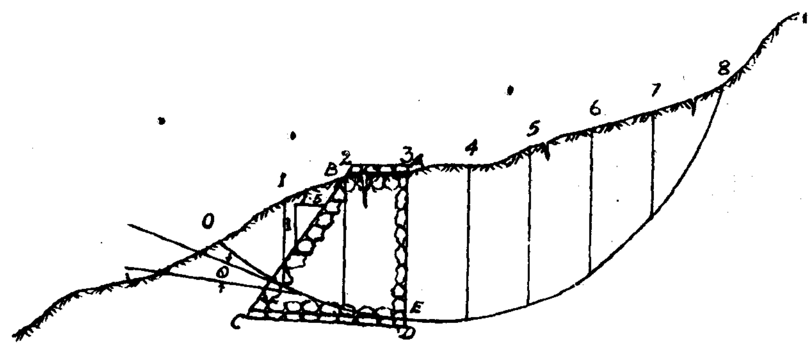

Fig. 12

Rock buttresses are most commonly used and, to avoid foundation failure, they are preferably constructed on solid rock foundation. The slip surface through the buttress is assumed to be a straight line. An approximate method of locating the buttress is that the tangent at the point of intersection of the slip surface and one edge of the buttress (E in the Fig. 12 makes an angle of less than 10° with the horizontal.

Now, horizontal shear through buttress:

$$P_R \cos\theta = r_B A_B \tan\phi_B \quad \ldots\ldots\ldots\ldots(3)$$

where θ = angle made by the tangent to slip surface with the horizontal at back of buttress.

r_B = unit weight of the buttress in lb per cu. ft.

A_B = area of the buttress in sq. ft.

ϕ_B = angle of internal friction for buttress rock.

For shear failure between the buttress and the foundation,

$$P_R \cos\theta = T_B A_B \tan\phi + C\frac{A_B}{h} + \frac{1.5\,h}{2} \qquad \text{.............(4)}$$

where h = height of buttress in ft.

In the above formula, the buttress is assumed to be constructed with one face vertical and the other on a 1.5 : 1 slope.

Base of buttress in that case is given by,

$$\frac{A_B}{h} \pm \frac{1.5\,h}{2} \qquad \text{.............(5)}$$

Referring to our examples and assuming the vertical face of the rock buttress to be in line 3 as shown in the Figure

$$\sum_3^8 T = 62{,}200 \text{ lb}$$

$$\sum_3^8 N = 132{,}800 \text{ lb}$$

$$L = 75 \text{ ft}$$

Therefore, $P_R = 1.5 \times 62{,}200 - 132{,}800 \times 0.1763 - 497.5 \times 75 = 32{,}500$ lb.

Assuming rocky foundation and taking

$$\phi_B = 30^\circ,\ T_B = 100 \text{ lb/cu. ft.}$$

since $\cos\theta = 0.985$, $h = 22$ ft, we get from equation (3)

$$A_B = \frac{32{,}500 \times 0.985}{100 \times 0.578} = 555 \text{ sq. ft.}$$

To be in a safe side over a conservative value obtained from the above, 10 per cent more may be added.

Thus we get $A_B = 384$ sq. ft. from equation (5)

$$\text{Top width of buttress} = \frac{555}{22} - \frac{33}{2} = 9 \text{ ft}$$

$$\text{Base width} = \frac{348}{22} + \frac{33}{2} = 42 \text{ ft}$$

F. S. (with reference to shear failure through the buttress),

$$= \frac{\sum_3^8 N\tan\phi + CL + \sum_0^3 N\tan\phi_B}{\sum_3^8 T + \sum_0^3 T}$$

$$= \frac{23{,}400 + 37{,}300 + 34{,}600}{84{,}000} = 2.01.$$

If, however, the bedrock is not encountered, then by applying formula (5), we get,

$$A_B = \frac{P_R\cos\theta - \dfrac{1.5\,h\,C}{2}}{r_B\tan\phi + \dfrac{C}{h}}$$

$$= \frac{32{,}500 \times 0.985 - \dfrac{1.5 \times 22}{2} \times 497.5}{100 \times 0.1763 + \dfrac{497.5}{22}} = 780 \text{ sq. ft.}$$

Thus, a larger area of buttress will be required. The top and base width may be worked out by application of formula specified above.

3. RETAINING WALL

Failure of a retaining wall may take place due to shear failure through the wall, foundation failure below the wall, shear failure between the wall footing and the foundation and due to overturning.

As before, for bedrock foundations we have,

$$P_R \cos\theta = W\tan\phi_W \qquad \text{............(6)}$$

and for soil foundations,

$$P_R \cos\theta = W\tan\phi_W + CL_W \qquad \text{............(7)}$$

where W = weight per foot of wall in pounds.

ϕ_W = angle of internal friction between wall and foundation.

L_W = length of slip surface below the wall in feet.

Referring to the Fig. 13 as before

$$P_R = 32{,}500 \text{ lb}$$

$$\cos\theta = 0.985 : \tan\phi_W = \tan 30^\circ = 0.578$$

Therefore from formula (6),

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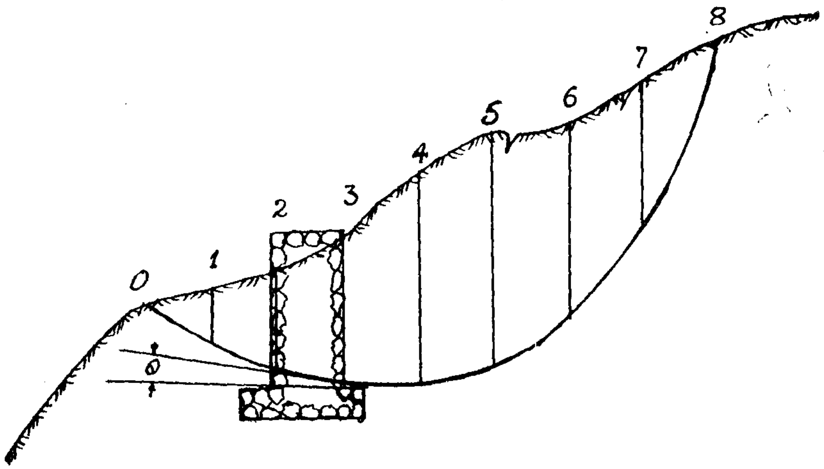


Fig. 13

$$W = \frac{32{,}500 \times 0.985}{0.578} = 55{,}500 \text{ lb}$$

Also from formula (7),

$$L_W = \frac{32{,}500 \times 0.985 - 55{,}500 \times 0.1763}{497.5} = 44 \text{ ft}$$

As in the preceding example, factor of safety in this case as well will be 2.01.

4. DESIGN OF BOULDER SAUSAGE

Though constructed to prevent landslides, the main purpose of a sausage is to resist earth pressure. In the cross section of the landslide area in mile 42 of Dimapur-Imphal road, a boulder sausage with top width 3 ft, bottom width 5 ft and height 6 ft was constructed. The sausage was designed as non-surcharged structure by excavating the earth to a distance of 11 ft for benching.

Assuming, for wet loose earth unit = 120 lb/cu. ft. and angle of repose = 16°. Unit weight of boulder = 120 lb per cu. ft. height of the sausage, h = 6 ft,

$$\therefore \quad \text{Earth pressure, } P = \frac{W_h{}^2}{2} \frac{(1 - \sin \phi)}{(1 + \sin \phi)} = \frac{120 \times 6^2}{2}$$

$$\times \frac{1 - 0.276}{1 + 0.276} = 1230 \text{ lb},$$

Taking top width to be 3 ft and the earth face to be vertical and dividing the trapezoidal section into a rectangle and a triangle as shown,

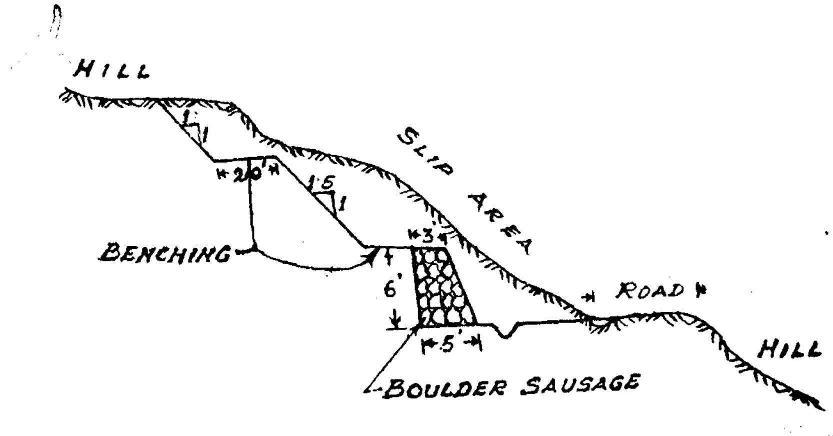


Fig. 14

base width of the triangle = x feet (say),

Now, weight of the triangular portion,

$$= W_1 = \frac{6x}{2} \times 1 \times 120 = 360x$$

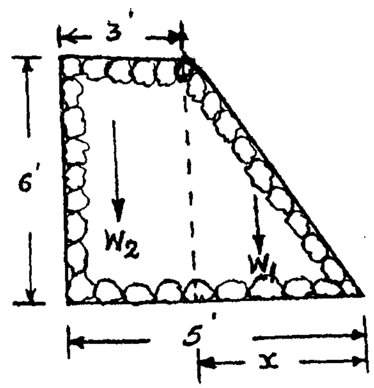


Fig. 15

and weight of rectangular portion,

$$W_2 = 6 \times 3 \times 1 \times 120 = 2160 \text{ lb}$$

Taking moments of the weights about the heel,

$$360x \times \frac{2x}{3} + 2160\left(x + \frac{3}{2}\right) = 1230 \times \frac{6}{3}$$

or, $x^2 + 9x + 13.5 = 10.3$ or $x^2 + 9x + 3.2 = 0$.

The value of x here comes out to be negative (instead of + 2) which shows that the proposed top width of 3 ft and bottom width of 5 ft are oversafe.

Annexure 8

INSTALLATION OF DRAINAGE STRUCTURE

This method is a direct rebalance of ratio between resistance and force. Two detrimental effects of water within a slide are that it increases the shear stress by its own weight and by increase of seepage forces, and it reduces the shear resistances of the material by increase of hydrostatic or pore pressure. Thus, by installation of a trainage structure, the forces that are contributing to the movement are decreased or the natural resistance to movement is increased.

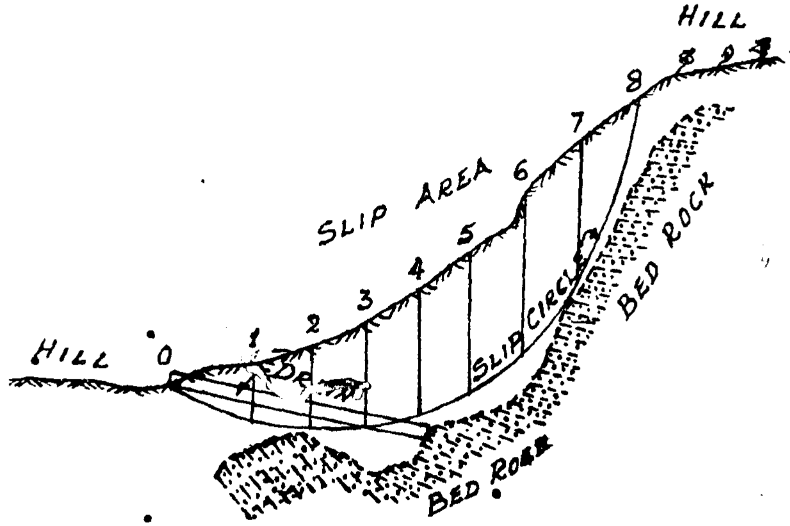


Fig. 16

Referring to the example of Annexure 6, for $\phi = 10°$, the average cohesion was computed to be 645.5 lb per sq. ft. when hydrostatic pressure was taken into consideration. If a drain is now installed in line 8 below the slip surface so as to lower the ground water table to coincide with the position of the slip surface, intergranular force will be eliminated.

Then,

$$F.S. = \frac{\Sigma (N - \mu) \tan \phi + CL}{\Sigma T}$$

$$= \frac{(193{,}000 - 0) \times 0.1763 \times (645.5 \times 100)}{84{,}000}$$

$$= \frac{34{,}000 + 64{,}550}{84{,}000} = 1.2$$

Thus, the factor of safety is increased by 0.2 on account of the installation of drainage structure.

However, when the drains are installed above the slipping plane but below the water table, the hydrostatic pressure will not be zero. As the water table will be lowered to an extent, the intergranular force will also be less than that calculated in Annexure 6. As a result, the factor of safety will be greater than unity (though less than 1.2, as calculated above).

Annexure 9

Benching of Slopes

When a *straight slope* cannot be cut sufficiently flat for reason of stability, benching is done by dividing the long slope into segments of shorter *slopes connected by benches.*

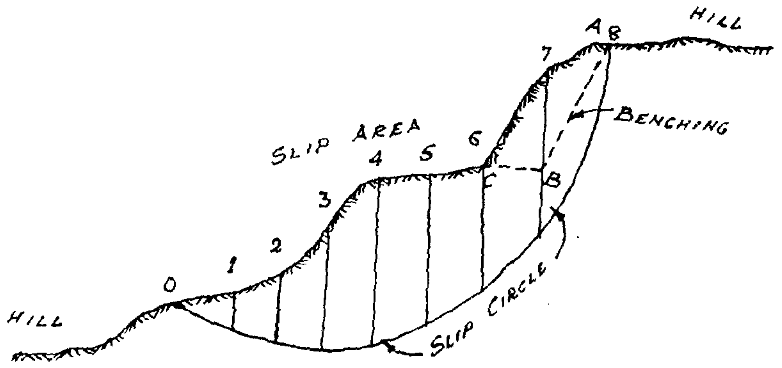


Fig. 17

In order to determine the effect on the stability, we assume that the bench ABC has been excavated. *As in* Annexure 6, computing the forces, we get :

Increments

	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8
Area in sq. ft.	77	180	245	275	274	238	190	120
Weight in Kips	10	23.5	32	35.9	35.8	31.1	24.8	15.7
Tangential (T)	6.05	9.2	6.5	1.6	8.7	14.5	17.0	13.2
Normal (N)	6.85	21.5	31.8	35.7	34.9	27.4	18.2	7.8

$\therefore \ \Sigma T = 77{,}700$ lb

$\Sigma N = 184{,}150$ lb

$\phi = 10°$

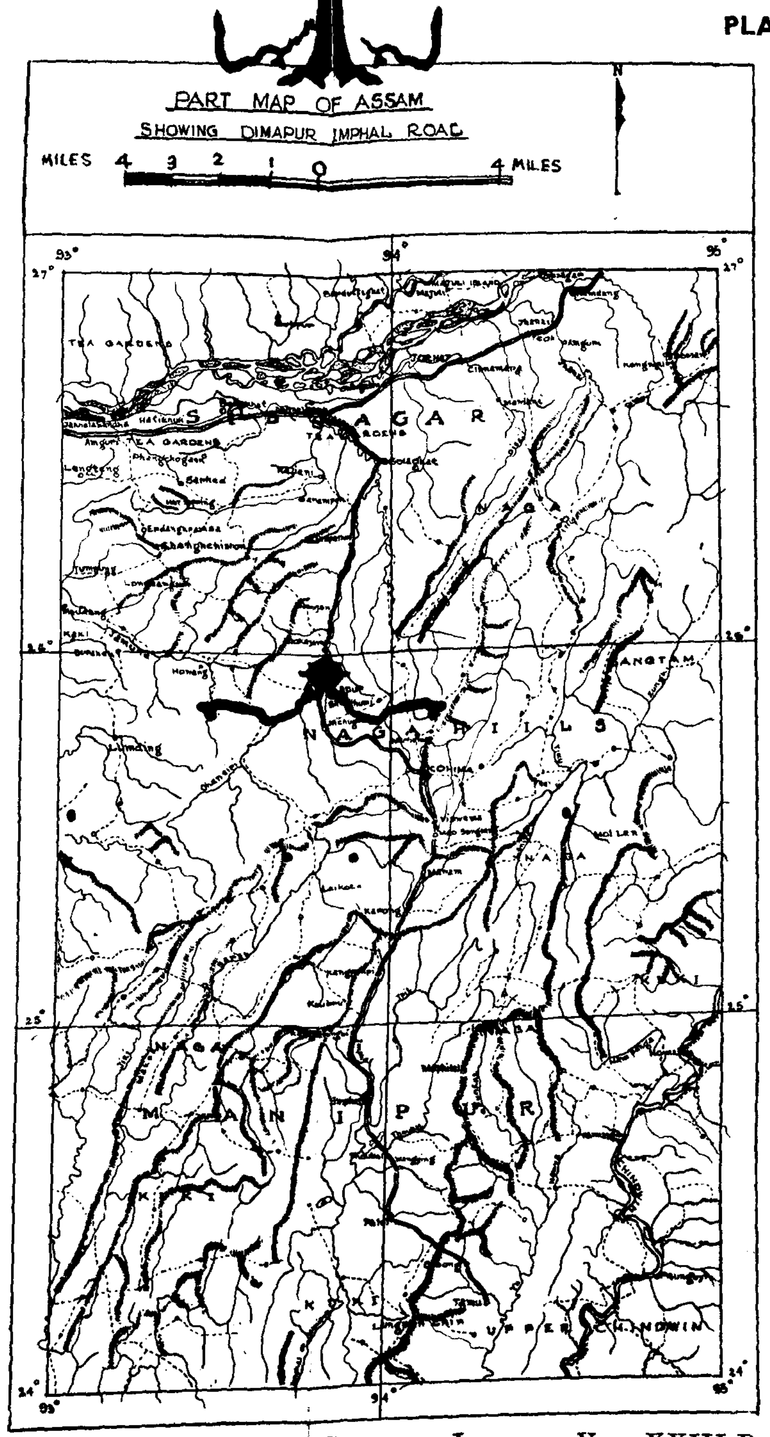
$C = 497.5$ lb per sq. ft.

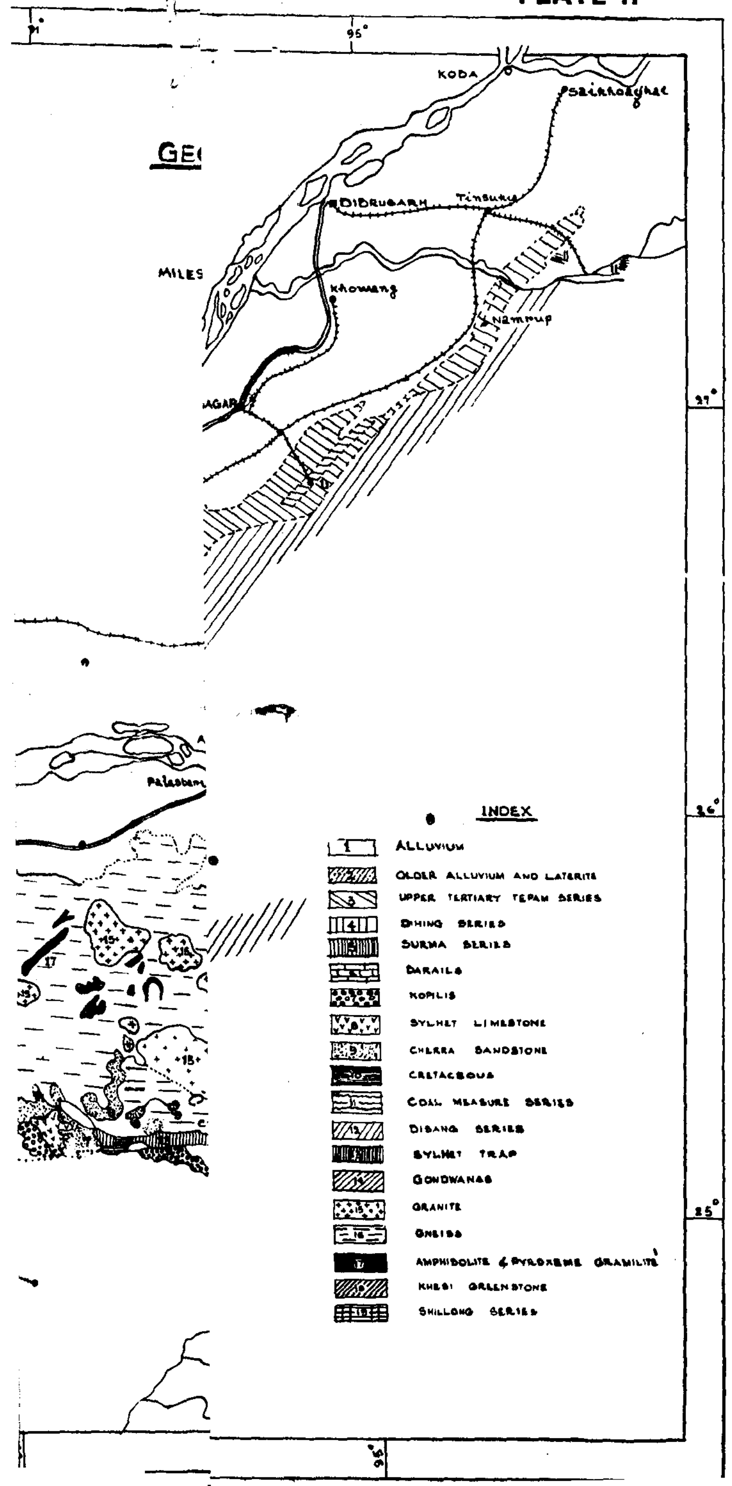
$L = 100$ ft

$$\therefore \ F.S. = \frac{184{,}150 \times 0.1763 + 497.5 \times 100}{77{,}700} = 1.16$$

Thus, the factor of safety is *increased* by 0.16.

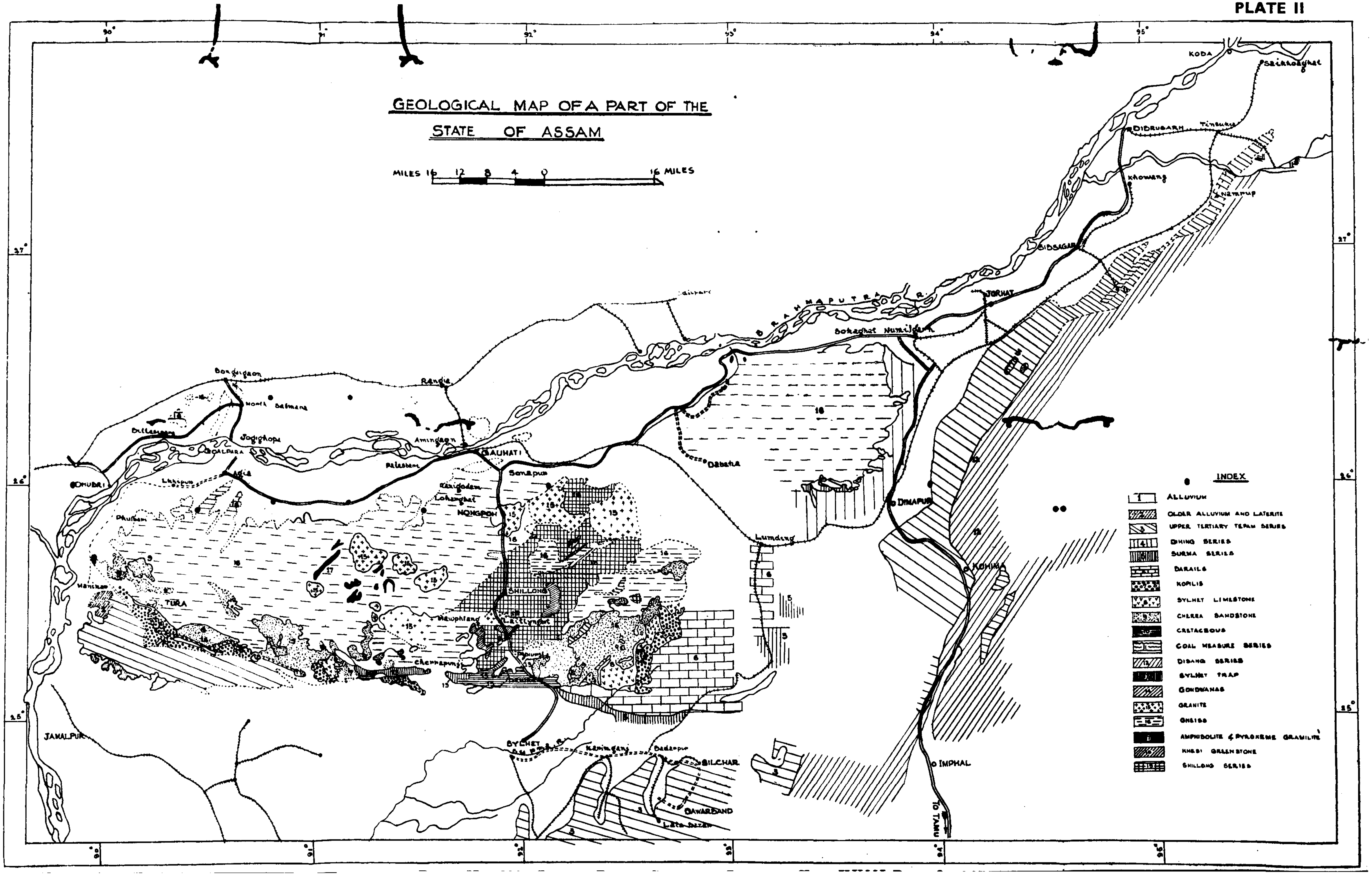
PLATE I





95°
KODA
DIBRUGARH
Tinsukia
MILES
Namrup
97°
26°
25°
INDEX
ALLUVIUM
OLDER ALLUVIUM AND LATERITE
UPPER TERTIARY TEPAM SERIES
DIHING SERIES
SURMA SERIES
BARAILS
KOPILIS
SYLHET LIMESTONE
CHERRA SANDSTONE
CRETACEOUS
COAL MEASURE SERIES
DISANG SERIES
SYLHET TRAP
GONDWANAS
GRANITE
GNEISS
AMPHIBOLITE & PYROXENE GRANULITE
KHASI GREENSTONE
SHILLONG SERIES

PLATE II



GEOLOGICAL MAP OF A PART OF THE STATE OF ASSAM
MILES 16 12 8 4 0 16 MILES
INDEX
ALLUVIUM
OLDER ALLUVIUM AND LATERITE
UPPER TERTIARY TEPAM SERIES
DIHING SERIES
SURMA SERIES
BARAILS
KOPILIS
SYLHET LIMESTONE
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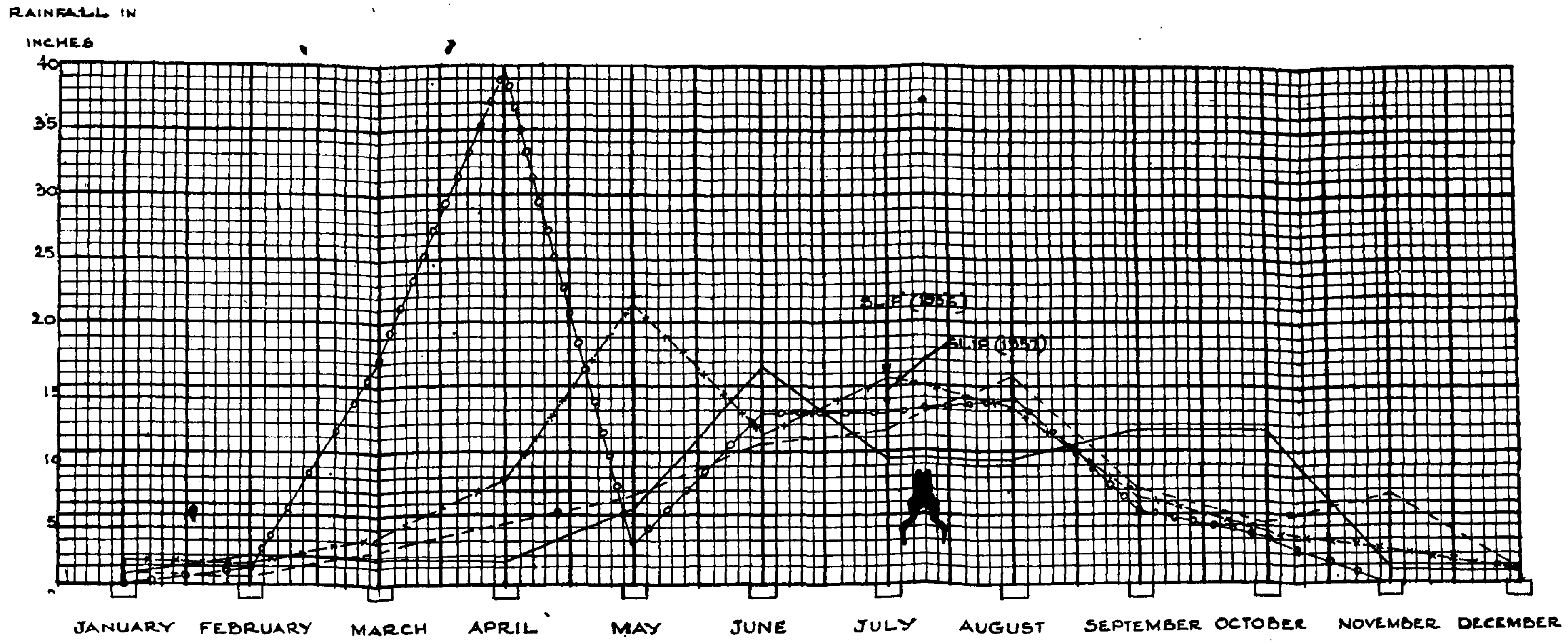
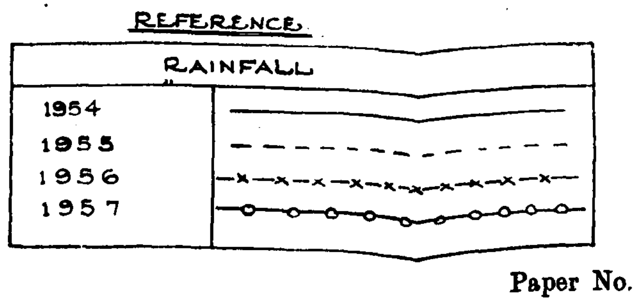
AMPHIBOLITE & PYROXENE GRANULITE
KHASI GREENSTONE
SHILLONG SERIES
BRAHMAPUTRA R.
KODA
DIBRUGARH
SIBSAGAR
JORHAT
GAUHATI
DHUBRI
GOALPARA
NONGPOH
SHILLONG
TURA
Cherrapunji
DIMAPUR
KOHIMA
IMPHAL
TO TAMU
SILCHAR
SYLHET
JAMALPUR
Lumding
Karimganj
Badarpur

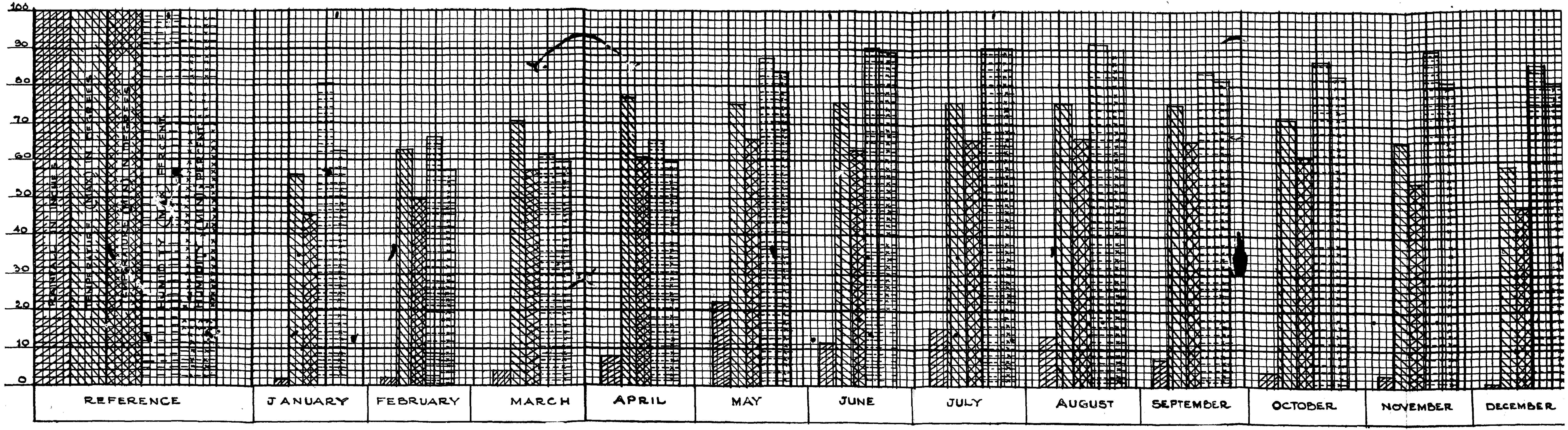


LANDSLIDES IN RELATION TO RAINFALL



NOTE:-
1. THE MAJOR LANDSLIDES OCCURED AFTER HIGHEST RAINFALL IN THE YEAR.
2. SLIPS INDICATE – MAJOR SLIDES IN MILE 34,40,41,42,65

PLATE IV



VARIATION OF RAINFALL-HUMIDITY AND TEMPERATURE AT KOHIMA DURING 1956 WHEN LANDSLIDES WERE SEVERE

PLATE V

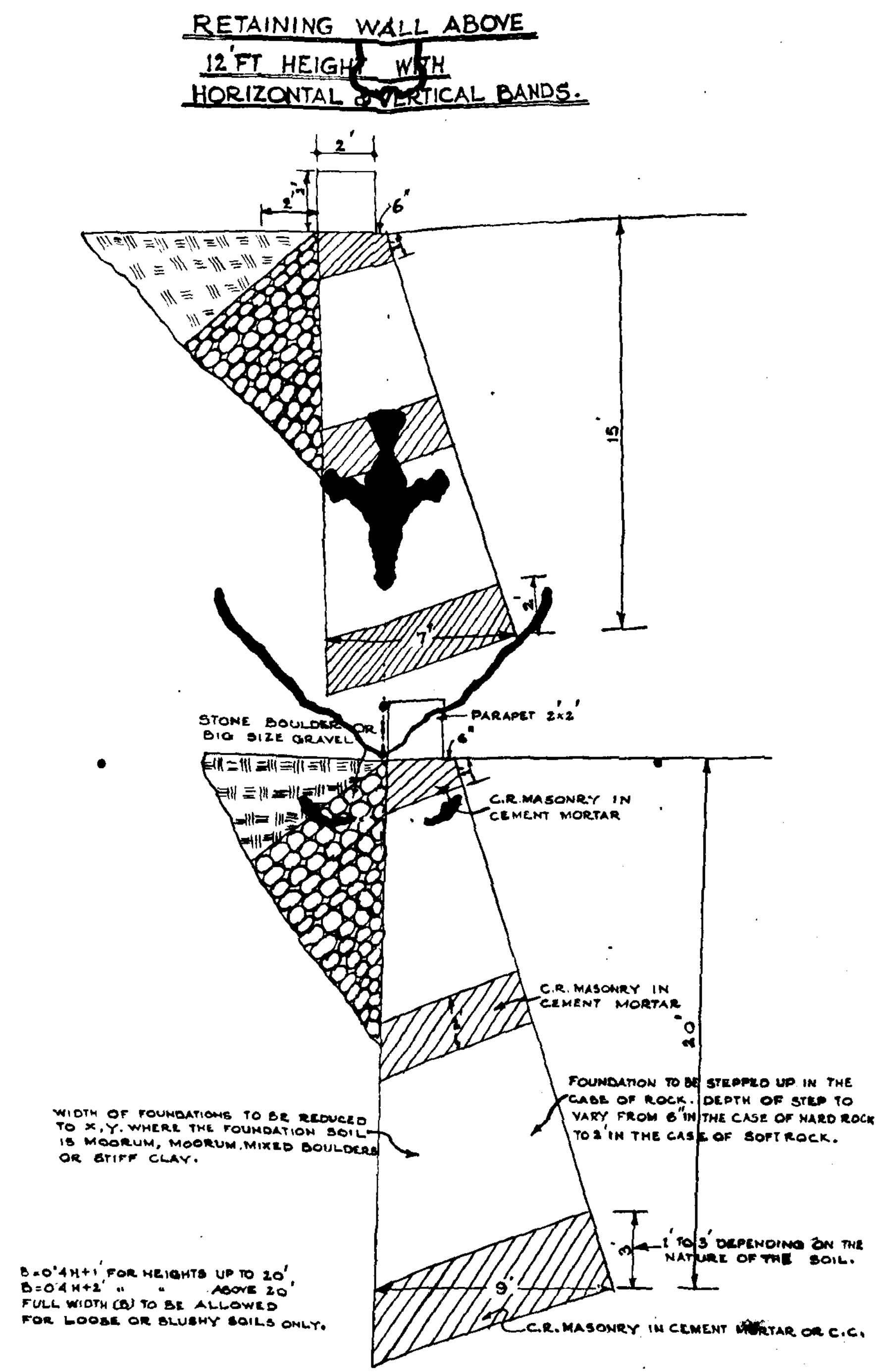

RETAINING WALL ABOVE
12 FT HEIGHT WITH
HORIZONTAL & VERTICAL BANDS.
2'
2'
6"
15'
7'
STONE BOULDER OR
BIG SIZE GRAVEL
PARAPET 2'x2'
6"
C.R. MASONRY IN
CEMENT MORTAR
C.R. MASONRY IN
CEMENT MORTAR
FOUNDATION TO BE STEPPED UP IN THE
CASE OF ROCK. DEPTH OF STEP TO
VARY FROM 6" IN THE CASE OF HARD ROCK
TO 2' IN THE CASE OF SOFT ROCK.
WIDTH OF FOUNDATIONS TO BE REDUCED
TO X, Y. WHERE THE FOUNDATION SOIL
IS MOORUM, MOORUM MIXED BOULDERS
OR STIFF CLAY.
1' TO 3' DEPENDING ON THE
NATURE OF THE SOIL.
9'
B=0·4H+1' FOR HEIGHTS UP TO 20'
B=0·4H+2' " " ABOVE 20'
FULL WIDTH (B) TO BE ALLOWED
FOR LOOSE OR SLUSHY SOILS ONLY.
C.R. MASONRY IN CEMENT MORTAR OR C.C.

PLATE VI

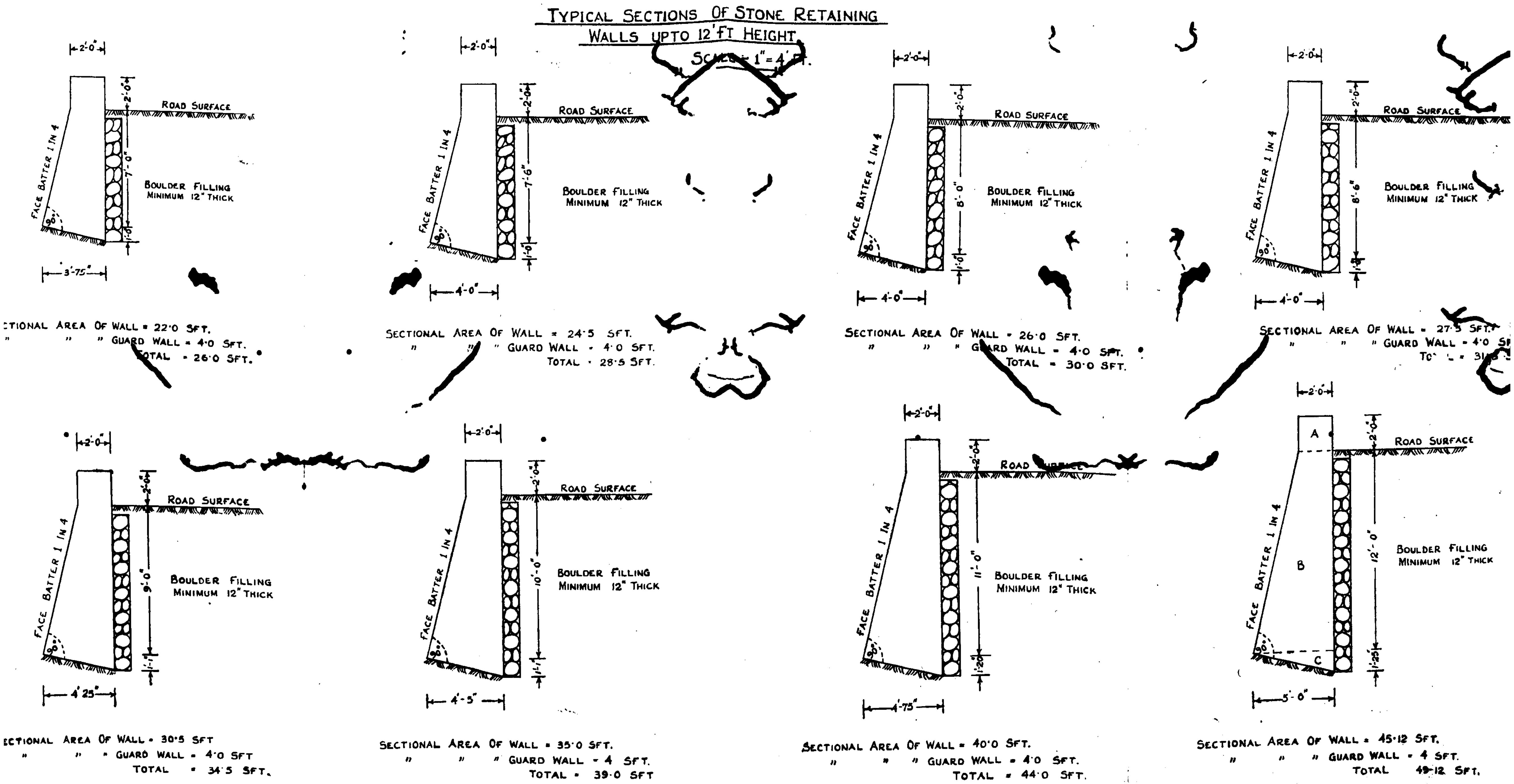
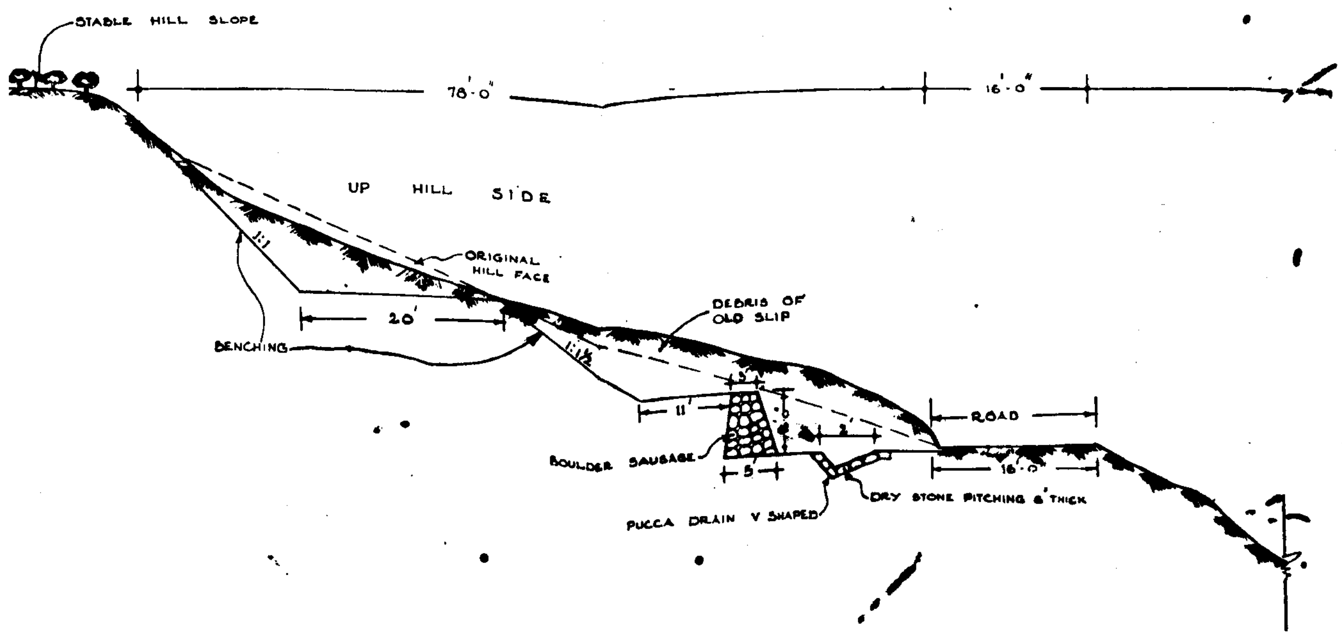
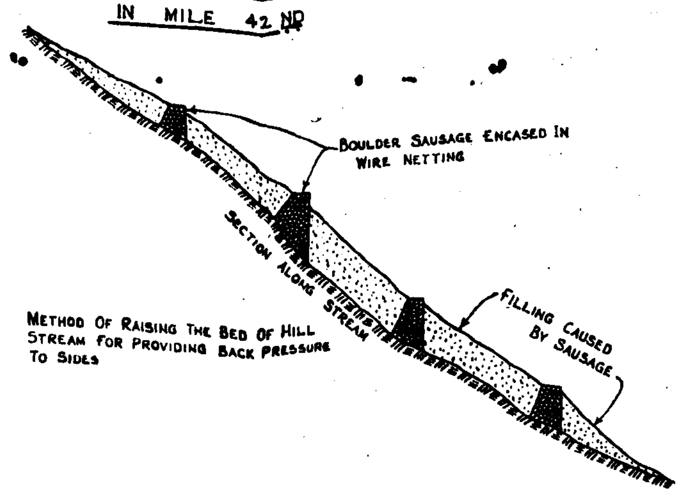

Typical Sections Of Stone Retaining Walls upto 12' Ft Height
Scale 1" = 4' Ft.
Road Surface
Boulder Filling Minimum 12" Thick
Face Batter 1 in 4
Sectional Area Of Wall = 22·0 Sft.
" " " Guard Wall = 4·0 Sft.
Total = 26·0 Sft.
Sectional Area Of Wall = 24·5 Sft.
" " " Guard Wall = 4·0 Sft.
Total = 28·5 Sft.
Sectional Area Of Wall = 26·0 Sft.
" " " Guard Wall = 4·0 Sft.
Total = 30·0 Sft.
Sectional Area Of Wall = 27·5 Sft.
" " " Guard Wall = 4·0 Sft.
Sectional Area Of Wall = 30·5 Sft
" " " Guard Wall = 4·0 Sft
Total = 34·5 Sft.
Sectional Area Of Wall = 35·0 Sft.
" " " Guard Wall = 4 Sft.
Total = 39·0 Sft
Sectional Area Of Wall = 40·0 Sft.
" " " Guard Wall = 4·0 Sft.
Total = 44·0 Sft.
Sectional Area Of Wall = 45·12 Sft.
" " " Guard Wall = 4 Sft.
Total 49·12 Sft.

PLATE VII



BENCHING OF SLOPE
IN MILE 42 ND



METHOD OF RAISING THE BED OF HILL STREAM FOR PROVIDING BACK PRESSURE TO SIDES

GENERAL REPORT

ON

ROAD RESEARCH IN INDIA

GENERAL REPORT ON ROAD RESEARCH IN INDIA*

CONTENTS

1. INTRODUCTION

It is heartening that increasing attention is being given to road research and as a result more and more laboratories are gradually coming into being in the country.

This is all for the good. There is one aspect of the development of research, however, which has to be borne in mind if any realistic approach is envisaged, viz., that applied research can emanate only from application itself. To make this possible, two conditions must be satisfied :

(a) The State laboratories, through field laboratories, under their direct control, should be made responsible for the testing of materials and for the control of quality of work on all road construction projects in the State.

(b) The existing knowledge on the design and construction of pavements should be utilized to the full, by making

*Compiled by the Director, Central Road Research Institute, New Delhi.

the State laboratory concerned responsible for giving the designed thickness, specifying the manner in which the locally available materials could be utilised for economy, and generally laying down specifications.

It would be a good thing if the State laboratories were to work in direct collaboration with the Central Design Office.

If the above conditions are satisfied, the State laboratories will work essentially as design, testing and control laboratories and will make themselves responsible for all the research problems emanating from actual construction. It is only by this process that the State laboratories while being immediately useful for economy and efficiency in construction, will side by side, steadily develop a sound applied research outlook based on the actual requirements of future road construction.

This General Report on Road Research in India has been compiled from the reports sent in by the various highway research organisations in the country. It is likely that there may be some omissions due to some of the reports not having been received by the time of compilation of this report It is, however, hoped that any such points will be brought up during discussions so as to make the report as comprehensive as possible.

After briefly reviewing the work done, salient points are given at the end of each section under the sub-head "Points for discussion" to focus attention on more important issues. This does not mean, however, that discussions on other points should not be taken up.

Points for Discussion

(i) Functions of laboratories.

(ii) Application of research.

I. DESIGN, CONSTRUCTION AND MAINTENANCE

2. RIGID PAVEMENTS

2.1. Design

General: It is encouraging to note that instead of using arbitrary standard mixes, concrete mixes are now being properly designed in some parts of the country. It is also interesting to note the various experiments being conducted on the use of admixtures in cement concrete mixes. In the field of structural design of concrete pavements, it will be instructive to watch the behaviour of the 12 in. c.c. pavement proposed by the Central Road Research Institute for the heavy duty Factory Main Road of Heavy Electricals (P) Ltd., Bhopal (M.P.).[1] The challenge of destructive effects of highly expansive black-soil, under high water-table conditions, has been met by providing bituminous envelope and thick moorum sub-bases. The pavement will cater to the unusually heavy transporters with multi-wheel assemblies (Gross wt. of 180 tons on 2 assemblies of 12 wheels each).

2.2. Construction and Maintenance

2.2.1. **General**: Except for the use of vibratory screeds for compaction of concrete in pavement construction, no improvement seems to have been made with regard to construction and maintenance of rigid pavements.

2.2.2. **Field Experiments: Pavement for Tracked Vehicles**

As reported last year, the College of Military Engineering, Kirkee[2] constructed a section of a roadway using the following specifications:

(i) Stone setts of 10 in. × 6 in. × 8 in. deep set on 1:2:4 concrete 8 in. thick.

(ii) Stone setts of 10 in. × 6 in. × 8 in. deep laid on 1:2:4 concrete 4 in. thick with minimum spacing in between.

(iii) 8 in. thick concrete pavement with a minimum compressive strength of 5,000 lb per sq. in. at 28 days.

(iv) Concrete slabs cast to 6 in. thickness and topped with 2 in. thick special mix using admixtures such as ironite and hardonate.

The base in all the cases consisted of rolled crushed rock or moorum.

Even though a limited number of tracked vehicles have used the test sections, at places of turning concrete has been cut by the blades of tracked vehicles to an extent of 1/8 in. in the sections consisting of cement concrete on top. In the areas where there are stone setts, no abrasion has taken place but some cracks have appeared in the mortar joints binding the stone-setts. The concrete section having 2 in. topping of ironite and hardonate has not fared any better than other concrete sections.

Grouted Cement Concrete Road Experiments

(i) As reported last year, an experimental test track of 2 miles length is being constructed near Faridabad on Delhi-Mathura Road with a view to compare the cost and performance of grouted concrete construction with that of premix concrete construction. The construction of the project sponsored by the Ministry of Transport and being executed by the Punjab P.W.D. with the technical assistance of the Central Road Research Institute and the Concrete Association of India is expected to be completed shortly. The slab thicknesses are 4 in. and 5 in. for each type of construction.

(ii) The Public Works Department of Uttar Pradesh[3] also tried a grouted concrete road experiment in mile 28 of Lucknow-Rae-Bareily Road in the year 1957, the slab thickness being 3 in. It has been reported that there was no saving in the cost of cement, coarse sand and aggregates as compared to 1 : 2 : 4 premix concrete. It is further stated that the grouted concrete pavement does not appear to be as strong as that constructed by the premix method and that it is already showing signs of distress.

In view of the reports received about the poor performance of grouted concrete pavement laid on Simla-Kalka road also, it is advisable to study the already laid experimental lengths in detail, before venturing further experiments on this type of construction.

2.2.3. Investigation of Failures

The investigation carried out by the Central Road Research Institute[4] on the failures of concrete pavements on expansive soils and reported last year has been completed. Based on the results of this investigation, the main cause of the failure is attributed to the excessive expansion and shrinkage of the subgrade consisting of black soil.

2.3. Laboratory Research

2.3.1. **Use of Admixtures in Concrete**: The work carried out at the Highway Research Station, Madras[5,6] has shown that incorporation of $\frac{1}{4}$ oz. to 1 oz. of Lissapol 'N' per bag of cement improved the slump and workability as tested with Wigmore consistometer without any effect on compressive strength. Dry mixes, however, did not show any improvement in workability. It was concluded that for the same slump, the water-cement ratio could be reduced, thus increasing the compressive strength with the incorporation of Lissapol 'N'.

The experiments conducted at the same Research Station have also shown that similar results are obtained with the use of Dehydol—25, a German product, at the rate of $\frac{1}{2}$ to $\frac{3}{4}$ oz. per bag of cement. It has also been shown that in 1 : 2 : 4 mix, there can be a saving of 10 per cent in the quantity of cement without loss of strength or workability.

2.3.2. Use of Pozzolanas in Concrete

The work carried out at the Central Road Research Institute has shown that surkhi prepared from Rayerkeri clay burnt at a temperature of 600°C. and pulverized to a fineness of 90 per cent passing B.S. sieve No. 200 can replace 20 to 25 per cent of cement by weight without any adverse effect on the compressive and flexural strengths of concrete. There was, however, a slight increase in the water demand due to the use of this surkhi. The use of coarser surkhi—56 per cent passing B.S. sieve No. 200—resulted in some reduction in the strength of concrete.

Work is in progress to carry out detailed investigations by using other types of surkhi.

The research work conducted at the P.W.D. Research Laboratory, Uttar Pradesh,[3] has shown that surkhi from third-class bricks gives better results than that from first-class bricks and that 10 per cent of the cement can be safely replaced by the use of the former, without appreciably affecting the properties of concrete. It has also been shown that shale, which is readily available in hilly parts of U.P., when calcined at a temperature of 700°C. gives more reactivity than when heated to 300 or 500°C.

2.3.3. Testing of Cement and Concrete

Tensile Test: Vibration compaction of cement-sand mortar briquettes for tensile strength, by using B.S.S. vibrating machine with a superimposed metallic weight on the specimen, has been tried at the Central Road Research Institute with a view to obtain better uniformity and reproducibility of the results. The preliminary data collected are encouraging and the work is in progress.

Testing of Concrete by Ultrasonic Pulse

Some work has been carried out by the Civil Engineering Department of the Indian Institute of Technology, Kharagpur,[7] on the use of ultrasonic pulse velocity method for predicting the trength of concrete specimens. The following broad observations iave been reported:

(i) Attempts to correlate dynamic modulus and compressive strength did not yield better results than those with wave velocity and compressive strength.

(ii) The use of velocity-strength curve, developed for one type of aggregate, for another type will lead to considerable error. It was also found that the dryness or saturation of specimens appreciably affected the results of evaluation of compressive strength from wave velocity.

(iii) For given types of aggregate and mix ratios, a mean curve can be developed between velocity and compressive strength. Compressive strengths can then be estimated from this curve for the same aggregates within an accuracy of 10-20 per cent.

(iv) The same mean curve can be used to predict the strength of cubes and cylinders. Similar tests are in progress at the College of Military Engineering, Kirkee.

Relation between W/C ratio and Compressive Strength of Concrete iade with Indian Cements

This relationship, which is the basis for all concrete mix esigns, is not available for Indian cements. The Central Road lesearch Institute has undertaken this study with the use of everal cements made by different factories in various regions of ndia. The work has been completed in respect of one cement nd further work with the use of other cements is in progress.

Measurement of Workability by Compacting Factor Apparatus

The work so far done in the U.P. P.W.D. Research Laboraory, Lucknow,[3] reveals that the compacting factor apparatus evised by the Road Research Laboratory (England) is more ensitive than the slump cone for determining workability of oncrete mixes.

2.3.4. **Membrane Curing of Concrete**[8] : A few liquid as well as astic membranes have been tried in the Central Road Research istitute. The laboratory results have indicated that certain astic membranes are superior to liquid membranes and comre favourably with water for curing of concrete. These memanes deserve field trials and may prove economical in arid gions of India.

2.3.5. **Alkali-aggregate Reaction:** Research work, conducted at the Central Road Research Institute has confirmed the concept of pessimum ratio between the alkali content of the cement and the reactive constituents in the aggregate.

It has also been shown that index glass manufactured in India can be used instead of imported pyrex glass as standard reactive material in such studies.

Studies on preventive measures for alkali-aggregate reaction carried out in the Central Road Research Institute have shown that the use of some surkhis, burnt at specified temperatures, is effective in preventing this reaction. This specified temperature is different for different clays and is usually higher than the optimum temperature for obtaining the maximum reactivity with lime.

2.3.6. **Gap Graded Concrete**[9] : The research work carried out at the Central Road Research Institute has indicated that some poorly graded sands not satisfying the accepted specification limits, can be used for making high quality concrete by deliberately introducing gaps in certain sizes of coarse aggregate.

The use of concrete designed on gap graded principle will permit the use of more types of sands which may be locally available.

2.3.7. **Joint Sealing Compound:** Further work on this problem has been reported by the College of Military Engineering, Kirkee [2] It is stated that the joint sealing compounds in some of the Indian airfields using jet aircrafts have developed blisters of varying sizes on mere exposure to the sun. In this connection it is interesting to note that rubberized bituminous compound manufactured by them has given partly satisfactory results in the laboratory as no blisters appeared when exposed to a temperature of 65°C. for a period of six hours. Further work is in progress.

2.4. **Prestressed Concrete:** A study has been taken up at the College of Military Engineering, Kirkee,[2] with a view to employ prestressed concrete for Military constructions. Experiments using a simple type of jack and locally developed anchorages are in progress.

2.5. **Points for Discussion :** (i) Comparative cost and efficiency of different methods of concrete road construction, including cement-bound macadam.

(ii) Use of poorly graded local sands for making high quality concrete by deliberately introducing gaps in certain sizes of coarse aggregate.

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8. "Use of Membranes for Curing of Concrete"—Jagus, P. J. & Sethi, K. L.—Indian Roads Congress Road Research Bulletin No. 5.

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3. FLEXIBLE PAVEMENTS

3.1. Bituminous Pavements

3.1.1. Design

(i) Performance of Pavements

In addition to the nature of the subgrade, the quality and thickness of each individual layer influences the success or otherwise of a bituminous or flexible pavement. Since failures often appear in the form of cracks on the black top surface, one should not conclude that only the riding surface has failed. Detailed investigations are necessary before the exact reasons for the cracks can be arrived at. Preliminary investigations,[1] carried out by the Central Road Research Institute reveal that the failures can be attributed generally to causes such as:

(a) Inadequate thickness of the pavement,

(b) Failure of the base course,

(c) Weathering of the binder, and

(d) Prolonged waterlogging.

During the investigation it was noted that field C.B.R. values together with McLeod's Chart can be used to determine as to whether the failure of the pavement was due to inadequate thickness or not. Based on the limited investigations done, it was also found that an approximate correlation between field C.B.R. and non-repetitive plate bearing test values within specific limits exists. It was further noticed that the required thickness of the pavement as dictated by the field test values is less conservative than the laboratory C.B.R. values.

(ii) Dense Carpets

A study of the design of dense carpets is being continued in the Central Road Research Institute.[2] It was observed that a definite correlation between the various tests does not always exist. Since the percentage of voids of the mixture is also one of the major criteria in the design, the void volumes were calculated based on bulk specific gravity as well as bulk impregnated specific gravity using only Delhi quartzite. From the limited number of mixtures studied, the void percentages were not very widely different, and it is felt that it may be so only in the case of highly absorptive types of aggregates. The work done at the B. & R. Research Laboratory at Karnal[3] suggests that a maximum stability within economic limit is obtained when the mixture has a grading such as given below:

Passing ½ in. and retained on ¼ in.—50 per cent.

Passing ¼ in. and retained on No. 10—15 per cent.

Passing No. 10—35 per cent.

Bitumen— 7½ per cent.

Binder contents of about 6½ to 7 per cent for a coarse graded mixture, 8 to 8½ per cent for a well graded one, and 9 to 9½ per cent for a fine mix are suggested. When studying the effect of filler on the density and stability of the carpets using limestone filler passing 40 mesh, it is reported by the same laboratory that the highest stability was obtained with 4 per cent filler content though the density of the mixture increased with increasing filler contents.

3.1.2. Construction and Maintenance

(i) Construction

It is encouraging to note that in some parts of the country mechanical pavers are being introduced for bituminous pavement construction. It is interesting to note that recent measurements taken with a bump integrator by the C.R.R.I. reveal that the unevenness index of the premix surfacing constructed by a mechanical paver without a seal coat was about 100 in. per mile whereas for the premix surface laid by manual labour, even with seal coat, it was of the order of 150 in. per mile.

(ii) Field Experiments

From the findings of the test track with 53 different specifications for stabilising fine desert sands, it was observed in the C.R.R.I. that the surfaces behaved quite satisfactorily under winter traffic but during summer heavy tracking and rutting was observed showing the instability of the bituminous sand during high road temperatures.[4] The sand used was such that about 80-85 per cent passed 100 mesh and was retained on 200 mesh. The binder used was 5 per cent of RC_3 cut-back. Hveem stability was about 12.5 and the percentage of voids in the laboratory sample was about 30 per cent. Further laboratory work is being undertaken to reduce the voids and increase the stability.

(iii) P.B.S. (Prefabricated bituminous surfacing)

During the last world war, a considerable amount of P.B.S. had been used for construction of roads, mainly in the eastern theatre of war. It was reported by the College of Military Engineering, Kirkee,[5] that two improved varieties of P.B.S. recently manufactured as per a new specification were received in the College with a view to assess their suitability for use in the field. After completion of laboratory tests on P.B.S. samples, a test track for the purpose of field trial had been laid and its service behaviour was under study. Field observations including field tests such as field density, moisture content and C.B.R. values of the pavement before and after the laying of P.B.S. have been carried out. From the observations so far made, it is reported that of the two, one variety exhibited excellent properties in regard to wear and tear and uniformity of surface texture.

3.1.3. Laboratory Research

(i) Stripping

An immersion wheel tracking test has been fabricated by the Central Road Research Institute and further work regarding the problem of adhesion using different types of aggregates is in hand. A similar machine has also been fabricated in the B. & R. Laboratory, Karnal. In the Madras Laboratory the study of the various stripping tests is being undertaken with a view to establish a firm standard for the determination of affinity of road stones to binder.

In the U.P. Research Station[6] both the plate test and Reidel and Weiber tests were adopted for a number of samples of stones and it was observed that the plate test was more severe, for which reason modifications of the test such as changing the duration of the test, etc., are being attempted. The laboratory data using Wet Fix and Adhevia T also showed an improvement in the adhesion of both tar and bitumen to road stones. A series of trial lengths of surface treatments has also been constructed in U.P. using Adhevia T, Wet Fix, white lime and cement with RT-2. During construction, wet conditions were created. From a preliminary observation, it was noticed that while the stretches with lime and cement are satisfactory, patches have developed in the other sections.

(ii) Low Temperature Tars

The low temperature tar that was obtained from the pilot plant of the Regional Research Laboratory of Hyderabad was analysed in the Central Road Research Institute and it was found to possess lower viscosity as compared to the high temperature tars. A small test strip was laid within the Institute's premises and it was found that the low temperature tar coats the aggregates much more easily. The strip is under observation from the point of view of weathering and cracking. Blending even with 50 per cent of a suitable bituminous binder has given quite satisfactory results in the laboratory.[7] Since the low temperature tar is being obtained only from the pilot plant, the uniformity in the grading and nature of the low temperature tar that is produced is to be ensured first. Further, the problem of changes that take place in the low temperature tar owing to storage has also to be surmounted.

(iii) Analysis of Binders

A detailed analysis of the various grades of tars obtained from different firms has been undertaken in the Central Road Research Institute with a view to see as to how far they conform to the modern trends in road tar manufacture. During the investigation,[8] it was found that a linear relationship between the E.V.T. and softening point of the tars exists. It was noticed that the absorption constant also bears a linear relationship with the E.V.T. of the tars in any one group of tars and as such the 'E' value can be used to distinguish between two tars possessing the same consistency characteristics. It was also found that the pitch-anthracene oil ratio for all the tars is not at a favourable level such as 3.5 as given in the German standards and the tars analysed did not have even a small quantity of heavy anthracene oil, which is claimed to improve the durability of tars. The work on the weathering and oxidation of road binders using the pressure oxidation chamber and the micro-viscometer is being continued.

The studies on the changes that take place in binders as a result of over-eating in the form of softening point and specific gravity for tars and penetration ductility and softening point for bitumens were undertaken in the U.P. Laboratory[6]; it was noticed that there is a marked difference in the physical properties as a result of over-heating.

(iv) Analysis of Road Stones

Road aggregates from different parts of the country are being studied in the Central Road Research Institute from the point of view of their physical and petrological properties with a view to specify the requisite properties for use in bituminous road construction.

Physical tests on aggregates from various[6] quarries in the U.P. State are being continued. Also, experimental stretches using four different types of aggregates for water-bound macadam

have been laid. For surface painting also, three types of aggregates, namely, brick, Jhansi stone, Birohi stone, were used and it was observed that the last stone gets crushed easily.

3.1.4. Points for Discussion

(i) The need for proper design of pavement thickness.

(ii) The necessity for proper design of expensive bituminous carpets.

(iii) The necessity for mechanisation of all bituminous work.

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3.2. Water-bound Macadam and Gravel Roads

3.2.1. **Water-bound macadam is the most prevalent** type of construction in this country. Depending upon the availability of road metal and soils, different types of materials are being used in different parts of the country. In addition to the aggregate interlock that we get in water-bound macadam, it is important to choose soils of proper PI as binding material for successful performance of the water-bound macadam road. This is more so in the localities where roads are subjected to bad drainage and waterlogging. Unfortunately, not much information is available on this aspect.

In the above context it is interesting to note the reported results of some of the investigations on road failures carried out by the Highway Research Laboratory, Madras.[1] The results of these investigations indicate that in the same water-bound macadam road the stretches where clay with high plasticity index was used as binder soil failed, whereas those stretches consisting of binder soil with less PI have been performing satisfactorily under the conditions prevalent in that particular locality.

Considering the magnitude of work involved in road building activities using water-bound macadam construction, it will be very useful if systematic studies are initiated to prescribe the nature of the binder soils that can be used in water-bound macadam construction.

3.2.2. **Stress Studies:** Even though some of the existing design methods, such as C.B.R., give the total superimposed thickness of the road crust for a particular wheel load and for a particular subgrade material, they do not differentiate between the various types of materials that can be used in the crust from the point of varying crust thickness. With a view to have better understanding on this aspect studies are in progress at the Central Road Research Institute to find the influence of different materials in the crust on the pattern of stress distribution in the road bed. The preliminary studies[2] so far conducted in this direction indicate the stress pattern changes in soil crust depending upon the moisture and density conditions.

The laboratory studies so far conducted at the Central Road Research Institute indicate that the low-grade aggregates such as kankar, laterite, etc., can be successfully used in road beds as bases and sub-bases. Work is in progress to suggest a set of tests together with the limits for successfully using such low grade aggregates in the road beds.

3.2.3. Points for Discussion

(i) Formulating specifications for the type of filler soil to be used in water-bound macadam construction.

(ii) Necessity for laying down tests and limits of test values for low-grade aggregates for use in bases and sub-bases.

(iii) Condition under which scientific grading and compaction of hard gravels for road construction is advisable.

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3.3. Soil Roads including Stabilization

3.3.1. **General:** A significant feature of this year's work has been the marked increase and emphasis on full-scale field trials in soil stabilization. Work on the construction of stabilized soil road lengths by Mehra's method, under the Pilot Plant Scheme, was initiated by the Central Road Research Institute.

Efforts towards improving grafting techniques in relation to the construction of stabilized soil roads have continued through the year. An important contribution to Applied Highway Engineering in the background of existing conditions has been the establishment of an indefatigable case for the need for proper compaction of the top 12 or 15 inches of the embankment to enable it to act as an effective part of pavement thickness.[1]

3.3.2. **Stabilization of Black Cotton Soil**: Continued laboratory experiments in the Central Road Research Institute [2] indicate that as a result of treatment of black cotton soils with lime, the C.B.R. value increases from as low as 3 to about 80 with the addition of 5 per cent lime. In the field trials carried out at Bhophe on Bhopal-Dewan Ganj Road, where a length of about 300 feet was laid using addition of both 5 per cent and 10 per cent lime, all improvement in bearing capacity was assessed by means of field C.B.R. tests on the soaked crust. The highest value of C.B.R. recorded in the field was 25 per cent and this is rather low compared with the laboratory values. This discrepancy between the laboratory and field values is presumably attributable to improper mixing done by disc harrows used in the work; it is, therefore intended to use the rotavator in future trials. However, in spite of such improper mixing it is possible to economise on the thickness of hard crust.

It is found that although lime treatment cuts down the plasticity of black cotton soils, still the resultant mixture is susceptible to moisture absorption with consequent reduction in strength. Experiments in the Central Road Research Institute conducted so far indicate that an addition of 4 per cent wood-tar to a 5 per cent lime-soil mixture serves to considerably reduce moisture absorption by the soil. The compressive strength of such a soil-lime mixture with the addition of wood-tar under saturated conditions is about 380 lb per sq. in. as compared to 200 lb per sq. in. for the same soil-lime mixture but without the addition of wood-tar.

3.3.3. **Mechanical Stabilization**: Most of the work done by the State laboratories during the period under review, consists of a number of full scale field trials applying existing specifications of stabilized soil road construction.

A road length of about 2 miles in stabilized soil was laid near Batala in waterlogged areas adopting specifications comprising bituminous envelope for capillary cut-off proposed by the Central Road Research Institute. The road has been in service for six months now, and is performing satisfactorily so far. The P.W.D. (Bihar) undertook construction of a sand-clay sub-base for a W.B.M. road on black cotton soil embankment. Construction of stabilized soil roads by Mehra's method, under the Pilot Plant Scheme of the Central Road Research Institute, was undertaken at several places such as Sriganganagar (Rajasthan), Basantpur (Bihar), etc.

Investigations by the Central Road Research Institute have clearly indicated the need for proper compaction of the subgrade so as to substitute part of the thickness of the pavement with processed soil in order to economise in cost of road construction. The C.B.R. value of the soil increases with increasing dry bulk density and consequently the required thickness of road crust on this improved soil decreases. It has been determined that it would be doubtlessly advantageous to compact 12 to 15 inches of the embankment subgrade immediately under the hard crust to a density of about 1.9 with the available compaction equipment at optimum moisture.

A cost analysis as worked out by the P.W.D. (M.P.)[3] with reference to their construction of soil stabilized road at Bhind is given below. The length of the road was 2 furlongs and width 10 ft. The base course was of 7 in. compacted thickness and the wearing course of 3 in. compacted thickness.

(a) Cost of unsurfaced soil stabilized road—Rs 36,800/ mile.

(b) Cost of surfaced 1 in. premix soil stabilized road —Rs 61,800/mile.

(c) Cost of WBM construction along the same stretch without 1 in. premix surfacing—Rs 63,200/mile.

3.3.4. **Chemical Stabilization**: With a view to improve the bearing capacity of black cotton soils, some experiments have been conducted by the Central Road Research Institute and it has been observed that the bearing capacity of black cotton soil intended for use as a stabilized soil wearing course improves considerably when treated with a small concentration of alum (0.5 per cent — 10 per cent). The soaked C.B.R. value of black cotton soils blended with sand and brick aggregates is found to increase from 20 per cent to 35 per cent by treatment with 1 per cent chemical. The work is only in its preliminary stages and further work is being conducted to know more about the chemistry of such a treatment.

In the U.P. P.W.D. Research Station at Lucknow[7] a tar emulsion using 75 per cent of RT-2, 20 per cent of kerosene and 5 per cent wax was prepared in the laboratory for making stabilized soil bricks. From a preliminary study in the same laboratory, it was observed that bitumen stabilized soil blocks containing percentages of 0.25 to 2 per cent of sodium sulphate and sodium carbonate disintegrated on immersion in water.

3.3.5. **Cement Stabilization**: A rapid method for the determination of cement content in a hardened soil-cement mass[4] has been developed in the Central Road Research Institute. The method is based on the estimation of the alkalinity of a soil-cement mixture.

3.3.6. **Stabilization with Lime and Molasses**: A small experimental length constructed by the B. & R. Research Laboratory, Karnal,[5] with stabilized soil using 2.5 per cent lime and 5 per cent molasses has been giving satisfactory behaviour for the last 18 months, whereas the untreated and cement-treated stretches have already shown signs of distress.

3.3.7. **Study of Subgrade Moisture**: As reported earlier, the Central Road Research Institute has carried out an investigation to determine the worst subgrade moistures in various parts of the country. As a result of preliminary work completed so far, it is observed that the degree of saturation in the subgrade during the most unfavourable seasons of the year is not necessarily 100 per cent as is generally assumed for the design of a flexible pavement, but varies over a wide range. The maximum probable degree of saturation varies from 20 per cent in extreme dry areas like Rajasthan to 100 per cent in areas abounding in black cotton soils and other areas subject to waterlogging in the alluvial plains. It has, therefore, been possible as a result of the above study to categorise India into different zones on the basis of subgrade moisture conditions for purposes of design.

It has been reported that investigations on subgrade moisture movement are also being conducted by the B. & R. Research Laboratory, Karnal. Subgrade moistures are being determined up to 3 ft depth and horizontally from the centre of the road crust to the centre of the berms at every 2 ft all the year round.

3.3.8. **Construction of Embankments on Marshy Ground**: The recommendations concerning the design and construcion of the 14 mile long Eastern Express Highway, Bombay, proposed by the Central Road Research Institute,[8] involve the application

of modern techniques of foundation engineering to some of the difficult problems in embankment design and construction.

A long section of the proposed highway is routed overlaying tidal flats randomly dissected by a number of creeks subject to tidal action. The high compressibility and the inadequate shear strength of the soft subsoil, the heavy traffic intensities, the periodic inundation of embankment due to tidal effects, the somewhat indifferent quality of materials available for embankment construction all—combine to present a challenging problem to the highway engineer.

To obviate the effects of detrimental settlement of the underlying soft clay layer, different methods of construction have been recommended for different zones. For most of the zones, some of the simplest possible recommendations have been made, such as "overbuilding" of embankment to compensate for the anticipated settlement and adopting "stage-construction", viz., allowing a specified lapse of time between completion of embankment construction and commencement of paving. Another inexpensive solution suggested, from the point of view of stability, has been that the rate of placement of embankment be so regulated as to take full advantage of the continually increasing strength of the subsoil below by virtue of its progressive consolidation due to the weight of embankment from above. In some stretches, use of lightweight materials, such as cinder, has been recommended so as to increase the factor of safety against displacement failures.

Other methods involving the use of explosives, partial or complete excavation of the undesirable material or displacement by overload surcharges, etc., were considered but were rejected as unsuitable either from practical or economic considerations. However, in some stretches, resort to some method of expediting the settlement becomes imperative, especially in view of the tight construction schedule. Installation of vertical sand drains has been recommended in such regions. The execution of this sand drain project will perhaps constitute one of the pioneering attempts in the field of applied Highway Engineering in India.

3.3.9. **Monographs**[6]: A review of some of the procedures used for finding the bearing capacity of subgrade soils has been brought out in the form of a monograph by the Central Road Research Institute.

3.3.10. **Points for Discussion**: (1) For purpose of strengthening the existing pavements, the feasibility of digging the road beds and reconstructing after compacting the subgrades wherever such treatment is more economical than direct superimposition.

(2) Remedial measures for roads in waterlogged areas.

(3) Possibility of introducing new techniques to improve the bond between stabilized soil wearing course and bituminous surface dressing.

(4) Construction of embankments on marshy ground.

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II. TRAFFIC AND TRAFFIC SAFETY

4. GEOMETRICS, ECONOMICS, STATISTICS AND SAFETY

General

It is encouraging to note the various studies conducted by the Central Road Research Institute and some of the State Laboratories in this branch of Highway Engineering during the year under review. During the period under report, the Central Road Research Institute carried out field studies with the co-operation of various organizations such as the Traffic Police, the Indian Institute of Public Administration and the Delhi State Administration.

4.1. Geometrics

4.1.1. **Design of Traffic Islands in Road Junctions**: A study[1,2] on the improvement of some road junctions in Madras State has been undertaken by the Highway Research Station, Madras. The junctions studied included 'Y' type intersections and their existing designs were considered inefficient.

As a result of this study, new designs of traffic islands have been recommended. These new designs along with proper road signs are considered more effective in segregating different types of traffic at the crossings with least hindrance.

4.2. Economics

4.2.1. **Public Transport Study**: As reported last year, a study of the Delhi transport system for public is being carried out by the Public Transport Study Group.

The Delhi public transport system faces a special problem during peak hours of morning and evening since 45 per cent of the employment in Delhi happens to be governmental. To solve this peak hour problem, staggering of office hours has been suggested[3] for the offices located in the office concentration area, i.e., the Central Secretariat and near about.

4.2.2. **Economics of Different Types of Highways**: In the present development programme, the problem that is frequently faced by the planners is to decide, on a particular location, under a set of known conditions, which type of pavement will be most economical in the long run taking into consideration the initial and recurring expenditure. With a view to have a general solution for this problem, studies have been initiated at the

Central Road Research Institute. It has been attempted in these studies to calculate the required thicknesses of different types of pavements such as flexible and rigid for variable conditions of subgrade, prevailing traffic and available materials, using the present day costs.

4.2.3. **Economics of Operational Costs of Different Types of Vehicles**: A complete and thorough knowledge of operating cost of different types of vehicles under actual operating conditions is needed for work in the field of highway economics. This information has many important uses, such as, the determination of the relationship between value of service and highway costs. These factors in turn influence highway location and enter into the determination of rates of motor vehicle taxation.

A study to determine the operating costs of all types of vehicles, including animal drawn vehicles, has been undertaken by the Central Road Research Institute.

4.3. Statistics and Safety

4.3.1. **Use of Sampling Techniques in Road Traffic Counts**: As reported last year, statistical sampling techniques are being employed to cut down the cost of taking annual traffic counts required for determining the average daily traffic on a particular road. In the design of the experiment, variations due to seasons, days of the week and hours of the day have been taken into consideration.

The traffic flow has now been counted for all the required number of days on two busy roads; one representing the predominantly office-going traffic and the other predominantly commercial traffic. The data[4] thus collected has been analysed this year. The results of the survey indicate that the sampling technique employed has proved highly efficient, and it further shows that substantial savings can be effected in the cost of survey by still reducing the period of counting.

The data obtained as a result of the abovementioned survey is being analysed with a view to evolve a suitable method for estimating daily traffic flow on a road by extremely short counts. The seasonal, weekly and daily patterns for different types of traffic have been worked out.

To meet the fast increasing traffic in and around Calcutta, the Road & Building Research Institute, West Bengal, is carrying out a detailed investigation. The report will come up as a Paper for discussion at the 23rd Session.

* Paper No. 208—Journal of the I.R.C., Vol. XXIII-2.

4.3.2. **Scientific Investigation of Road Accidents**: Scientific analysis and recording of road accidents is being studied by the Central Road Research Institute in continuation of the investigations reported last year.

Analysis of accident data in Delhi shows that the number of accidents in Delhi is increasing from year to year. The number of accidents rose from 1,127 in 1956 to 1,893 in 1957—an increase of 70 per cent. The number of persons killed in Delhi road accidents in 1956 and 1957 is 98 & 136 respectively, whereas the number of persons injured has increased from 731 to 962 during the same period.

The road-accident investigation system of the Traffic Police has also been studied and it has been proposed to centralize the accident records. Preliminary arrangements have been made to train up a special accident investigation squad of the traffic police in the Central Road Research Institute. Both the legal and the scientific implications of the problem will receive attention in these investigations.[5]

4.3.3. **Use of Plastic Markers**: Plastic markers have been fixed by the Central Road Research Institute at one of the pedestrian crossings in New Delhi, and their field performance is under test. During the last six months some of the markers have cracked, and the reasons for cracking are under investigation. Presumably the cracking is due to the mixed traffic including iron-tyred bullock carts.

The plastic markers at present used are imported from the U.S.A. Arrangements have been made with an Indian firm to manufacture these plastic markers, thereby saving the much needed foreign exchange.

4.3.4. **Road Users' Behaviour Study**: A new road sign, "Cross Walk", has been developed[6] by the Central Road Research Institute for channelizing the pedestrians to use the pedestrian crossings at road junctions. This road sign has proved very effective under field conditions. Arrangements are now being made to induce an Indian firm for its large-scale manufacture.

Points for Discussion

1. Feasibility of staggering office hours in office concentration areas in big cities for easing traffic congestion.

2. Development of new 'sampling techniques' and 'short-counts' that save time and money for assessment of traffic volume on our highway and city street systems.

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"A STATISTICAL EXAMINATION OF THE CLASSIFICATION SUGGESTED FOR THE BLACK COTTON SOILS OF INDIA"

By

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1. INTRODUCTION

1.1. The Central Road Research Institute proposed a classification of black cotton soils of India, which was published in the Indian Roads Congress Journal, Volume XVIII-2, 1953. The classification was based on 210 samples collected from different places. These samples were subjected to the following tests: liquid limit, plastic limit, fraction coarser than 200 U.S. standard sieve, shrinkage limit, field moisture equivalent and hygroscopic moisture.

1.2. It was shown that limits of A-7 group 1942 version of the P.R.A. system completely accommodated all the samples, and sub-grouping of A-7 on the basis of demarcation line for A-7-5 and A-7-6 (P.I. = LL-30) did not help in further sub-division of black cotton soils as the properties of soils above and below this line were not very much different. The sub-division of the black cotton soils was, therefore, suggested on the assumption that truly fat clays had generally liquid limit above 65. The samples in A-7 were, therefore, divided into two groups by drawing a demarcation line at liquid limit 65. For the limits of A-7 group, soils below 65 liquid limit will have maximum plasticity index of 42. The samples of higher liquid limit were further sub-divided on the basis of plasticity index below and above 42, thereby resulting into three main subgroups as shown in Fig. 1, viz.,

(1) Soils with liquid limit below 65 and plasticity index below 42—(a)

(2) Soils with liquid limit above 65 and plasticity index below 42—(b)

(3) Soils with liquid limit above 65 and plasticity index above 42—(c)

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1.3. This classification was checked up with the help of group indices derived by extending the existing lines of present group index chart, maintaining the same slope. The group index is the function of liquid limit, plasticity index and fraction

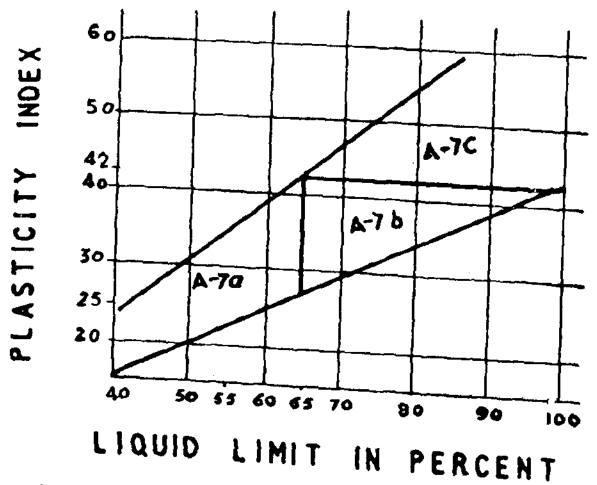

Fig. 1. **Showing the limits of sub-groups of the black cotton soils, A-7a, A-7b and A-7c, based on the values of liquid limit and plasticity index**

coarser than 200 U.S. standard sieve. It was found that the suggested soil group (a) had group index between 20-30, that of group (b) between 30-40 and of group (c) between 40-50 with a few minor exceptions.

1.4. It will, however, appear that the classification of the black cotton soils as suggested earlier by the Central Road Research Institute was based on an arbitrary assumption that truly fat clay soils should have a minimum liquid limit of 65 and further that properties of soil for the same liquid limit will be different for a corresponding plasticity index of above and below 42. Forty-two was again an arbitrary figure because a line dividing A-7 group vertically at 65 liquid limit touched the plasticity index at 42.

1.5. Though the above classification was supported by a modified form of original group index method, yet there were many samples which fell in A-7a, A-7b or A-7c, but did not have the corresponding group index values of 20-30, 30-40, and 40-50. These discrepancies were not explained. The object of this Paper is to examine the data statistically and find out the discriminating power of the characteristics for the suggested three sub-groups of the black cotton soils of India. The analysis has further been extended to form three groups and compared with the suggested classification.

2. STATISTICAL ANALYSIS

2.1. In order to test the discriminating power of the characteristics, discriminant analysis extended to three groups has been performed. The principle involved in this method is that the discrimination of a sample, which is identified by two or more characteristics, into three sub-groups, viz., a, b and c, will be good, if the ratio of the variation between the groups to the variation within the groups is a maximum. This will involve setting up a discriminant function of the type $Z = \lambda_1 X_1 + \lambda_2 X_2 + \lambda_3 X_3$ where X_1, X_2 and X_3 are the values of liquid limit, plasticity index and fraction passing 200 sieve, and λ_1, λ_2 and λ_3 are the corresponding weights. By analysis of variance technique, the values of Z for various samples are tested and if found significant, then discrimination between groups is said to be successful.

2.2. The total number of samples tested for their physical properties was 210. Out of these samples, the classification of 9 samples was found questionable on the basis of group index values. The discriminant analysis has thus been conducted by taking only 201 samples into consideration. They are then arranged into three groups formed on the basis of group index values. Using the values of these three groups the discriminant function—

$$Z = 1.2170\ X_1 + 1.8430\ X_2 + X_3$$

has been arrived at, where X_1 represents liquid limit, X_2 = plasticity index and X_3 = fraction passing 200 sieve. The equation shows very clearly the relative values of the three characteristics in distinguishing the suggested three sub-groupings. The plasticity index has the highest discriminant power as compared to liquid limit and fraction passing 200 sieve. By the analysis of variance test, the discriminating power of the equation has been established to be highly significant. As a result, it can be said that samples of black cotton soils are well classified by the suggested method. However, the classification of a few samples must be reconsidered. This will be evident when grouping the soil samples according to the discriminant function, the method of which is explained in the following para.

2.3. The values of Z using the means of liquid limit, plasticity index and fraction passing 200 sieve for the three groups are calculated below:

$$Z_1 = 1.2170 \times 57.63 + 1.8430 \times 28.97 + 83.11 = 206.6372$$

$$Z_2 = 1.2170 \times 71.80 + 1.8430 \times 36.33 + 89.39 = 240.1522$$

$$Z_3 = 1.2170 \times 84.88 + 1.8430 \times 48.23 + 92.22 = 284.4069$$

The standard error of the group means (σ) is the square root of the variance derived from the analysis of variance test. The limits set by the standard error of the group means will not completely decide the classification of the black cotton soils into the suggested three sub-groups, viz., A-7a, A-7b and A-7c, as most of the samples will fall outside these limits. In such cases the classification will be decided by the nearness of the individual Z value to the group means. Thus, for practical purposes the limits will be formed by equal distribution of the difference between $Z_3 - Z_1$ to the group means. By adopting this method the working limits for each of the sub-groups come as follows:

Group I (A-7a)	187.1950	—	226.0798
Group II (A-7b)	226.0799	—	264.9644
Group III (A-7c)	264.9645	—	303.8493

2.4. This method has been applied to all the samples and each one of them excepting sample Nos. 15, 27, 40, 50, 66, 73, 103 107, 113, 132, 150 and 188, i.e., about 6 per cent of the total samples subjected to discriminatory analysis, conform to the suggested classification by group index method as reported in Tables 1, 2, and 3. Out of these 12 samples, the Z value for sample No. 113 is 256.8422 and thus should be classified under A-7b instead of A-7c. Again Z values for sample Nos. 40, 66, 73 and 188 lie below the lower limits of A-7a group and Z values for sample Nos. 15, 27, 50, 103, 107, 132 and 150 lie outside the upper limits of group A-7c and thus require to be classified under new groups.

3. DISCUSSION OF RESULTS

As mentioned in the earlier paragraph, the classification of 12 samples, i.e., about 6 per cent of the samples of black cotton soils, based on the values of group index obtained from the modified group index chart, is questionable. Out of these 12 samples, one sample, viz., No. 113, needs a proper placement within the suggested sub-groups, whereas sample Nos. 40, 66, 73, 188, 15, 27, 50, 103, 107, 132 and 150 require to be classified into new groups. In sample Nos. 15, 27, 50, 132, 107, 103, and 150, it is noticed that the value of liquid limit is about 100. The samples involved are only 7, and a new group may not be necessary to accommodate these few samples. As regards sample Nos. 40, 66, 73, and 188 for which also a new group has been suggested, it is stated that all these samples fall near the extreme lower limit of A-7a group suggested on the basis of liquid limit and plasticity index values. These samples have very low liquid limit and plasticity index and, therefore, cannot be called typical black cotton soils.

4. CONCLUSION

The analysis of the data attempted in this Paper indicates that the black cotton soils of India are well classified by the suggested method published already. The misclassification is only to the extent of 6 per cent of the total samples investigated.

5. ACKNOWLEDGMENTS

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TABLE 1

Physical properties of the soils from Hyderabad State

Serial No.	Sample No.	Liquid limit per cent	Plasticity index	Fraction passing 200 sieve per cent	Classification group	Group index
1	1	80.36	48.95	93.30	c	45
2	2	89.76	47.62	87.32	c	44
3	3	71.60	37.89	87.61	b	37
4	4	81.71	48.18	88.15	c	44
5	6	86.05	39.85	81.40	b	38
6	7	73.23	41.23	83.00	b	38
7	8	62.50	36.79	77.14	a	29
8	9	82.72	50.02	92.80	c	46
9	10	54.04	18.01	91.80	a	20
10	11	69.81	30.87	98.00	b	35
11	14	70.30	33.97	94.89	b	35
12	15	100.00	58.07	95.47	c	48
13	16	75.48	34.74	87.70	b	35
14	17	86.63	54.49	90.70	c	46
15	18	80.19	40.31	95.40	c	43
16	19	84.55	44.05	92.31	c	46
17	20	90.55	49.99	96.88	c	48
18	21	84.02	45.96	96.35	c	46
19	22	83.28	44.02	95.42	c	46
20	23	83.45	46.87	99.07	c	49
21	24	65.96	32.81	93.02	b	34
22	25	88.55	47.05	96.60	c	48
23	26	60.19	24.65	85.20	a	25
24	27	100.80	58.24	83.07	c	43
25	28	65.40	30.73	83.26	a	28
26	29	65.47	24.98	84.60	a	25
27	30	66.35	32.42	92.80	b	34
28	31	69.30	36.29	86.57	b	34
29	32	74.48	44.93	91.35	c	42
30	34	62.03	38.12	76.22	a	29
31	35	84.02	53.84	90.85	c	45
32	36	57.15	34.53	77.87	a	27
33	37	89.22	55.22	89.91	c	45
34	39	49.60	29.38	79.65	a	22
35	40	49.51	26.01	75.80	a	20

TABLE 1—(*Contd.*)

Serial No.	Sample No.	Liquid limit per cent	*Plasticity* index	Fraction passing 200 sieve per cent	Classification group	Group index
36	41	77.41	46.29	85.35	c	42
37	42	64.46	36.22	69.73	a	26
38	43	58.14	31.89	68.54	a	22
39	45	85.49	54.13	95.35	c	48
40	46	90.17	51.86	92.67	c	46
41	47	85.59	47.39	86.17	c	43
42	49	72.19	42.00	84.32	b	38
43	50	91.35	58.52	87.13	c	44
44	51	79.54	41.90	90.81	c	41
45	52	77.76	43.03	92.55	c	43
46	53	83.42	46.49	83.80	c	43
47	54	56.54	34.85	63.30	a	20
48	55	80.14	44.82	87.10	c	43
49	56	87.94	54.34	87.87	c	44
50	57	73.43	42.00	89.70	b	40
51	58	72.58	39.87	91.00	b	4[illegible]
52	59	79.68	41.67	88.75	c	[illegible]1
53	60	63.57	30.96	69.20	a	22
54	62	53.07	27.77	75.30	a	22
55	64	51.11	29.98	77.00	a	24
56	65	58.43	29.06	82.26	a	26

TABLE 2

Physical properties of the soil from Bombay State

Serial No.	Sample No.	Liquid limit per cent	Plasticity index	Fraction passing 200 sieve per cent	Classification group	Group index
1	66	42.31	22.87	83.19	a	18
2	69	47.76	25.93	85.80	a	22
3	71	55.92	30.59	68.17	a	20
4	72	58.80	27.49	81.88	a	25
5	73	51.47	22.28	80.20	a	19
6	74	67.61	41.25	90.80	b	38
7	75	64.05	23.71	80.71	a	23
8	76	62.18	26.67	94.20	a	29
9	77	81.01	40.59	97.10	c	44
10	78	73.92	37.31	87.32	b	36
11	79	81.20	50.95	93.30	c	46
12	80	62.62	27.90	72.86	a	22
13	81	56.23	30.62	87.46	a	28
14	82	73.28	45.92	85.81	c	44
15	85	88.38	49.86	93.60	c	46
16	87	65.63	27.21	91.55	a	29
17	88	76.32	42.65	95.62	c	44
18	89	80.44	42.42	97.31	c	45
19	90	75.63	39.81	87.79	b	39
20	91	66.57	31.79	95.73	b	34
21	92	88.34	49.93	89.16	c	45
22	93	72.22	39.56	90.30	b	39
23	94	79 13	35.14	95.97	b	40
24	95	81.69	41.51	91.60	c	43
25	96	75.92	35.92	83.08	b	35
26	97	89.43	46.88	97.62	c	48
27	98	67.65	29.78	94.11	b	33
28	100	68.80	31.76	94.80	b	34
29	102	65.37	29.06	70.94	a	22
30	103	91.74	54.35	97.51	c	48
31	104	70.96	29.95	94.70	b	34
32	105	84.00	41.82	94.90	c	44
33	106	64.02	31.76	80.87	a	29
34	107	95.36	53.98	98.29	c	49
35	108	83.83	42.58	94.34	c	46
36	109	57.63	24.13	89.56	a	24
37	110	72.59	32.47	96.10	b	36
38	111	74.06	43.76	96.26	c	44
39	112	66.06	32.10	91.70	b	32
40	113	78.40	37.01	93.22	b	40
41	114	73.19	41.09	86.70	b	40
42	115	89.84	37.39	77.40	b	32
43	116	78.40	37.01	93.22	b	40
44	117	83.88	43.56	90.00	c	44
45	118	75.85	29.53	84.00	b	32

TABLE 2 (*Contd.*)

Serial No.	Sample No.	Liquid limit per cent	Plasticity index	Fraction passing 200 sieve per cent	Classification group	Group index
46	119	67.86	39.50	89.70	b	37
47	120	71.88	40.38	88.80	b	39
48	121	67.16	32.02	91.80	b	33
49	122	80.50	42.80	95.15	c	45
50	123	71.43	39.03	90.45	b	38
51	124	61.64	25.64	84.95	a	25
52	125	62.39	23.72	91.15	a	26
53	126	75.96	36.43	93.00	b	39
54	128	78.68	41.00	85.00	b	39
55	130	94.95	48.92	93.05	c	47
56	132	101.88	51.88	93.50	c	47
57	133	69.20	32.20	85.75	b	31
58	134	69.98	34.27	90.00	b	34
59	136	64.38	30.17	77.13	a	26
60	137	80.07	43.67	87.60	c	43
61	138	78.84	33.43	91.90	b	37
62	139	74.14	40.46	91.80	b	40
63	140	69.15	26.65	94.70	b	32
64	141	56.37	27.37	84.40	a	25
65	143	60.09	25.19	88.60	a	25
66	144	56.15	23.45	81.30	a	21
67	145	67.00	33.10	90.00	b	33
68	146	66.90	28.00	96.00	b	37
69	148	56.87	31.77	76.80	a	25
70	149	60.12	28.42	95.60	a	30
71	150	90.09	60.79	86.90	c	44
72	151	56.72	28.15	90.00	a	27
73	152	70.31	39.44	81.50	b	35
74	153	76.42	40.84	89.65	b	40
75	154	63.52	28.23	83.05	a	27
76	155	98.44	52.29	94.25	c	47
77	156	79.42	34.01	88.90	b	37
78	157	71.05	34.31	88.15	b	34
79	158	71.17	30.36	86.00	b	31
80	159	86.74	36.74	83.40	b	37
81	160	77.51	36.01	93.20	b	39
82	161	72.07	38.67	82.00	b	35
83	162	62.32	30.42	89.35	a	29
84	163	71.16	33.36	86.35	b	32
85	164	71.59	39.99	89.75	b	39
86	170	67.44	31.17	96.25	b	35

TABLE 3

Physical properties of the soils from Madhya Pradesh State

Serial No.	Sample No.	Liquid limit per cent	Plasticity index	Fraction passing 200 sieve per cent	Classification group	Group index
1	172	66.08	32.90	94.40	b	34
2	173	67.04	42.79	73.46	b	34
3	174	50.06	29.99	85.40	a	27
4	175	60.44	31.56	82.60	a	28
5	176	61.44	33.36	83.60	a	30
6	177	53.30	30.51	77.10	a	23
7	178	58.49	28.64	94.31	a	29
8	179	56.24	33.96	87.50	a	29
9	180	72.19	41.35	92.49	b	40
10	181	73.10	37.39	98.15	b	40
11	182	66.37	38.10	95.00	b	38
12	183	49.22	28.20	80.38	a	25
13	184	65.98	38.89	96.09	b	39
14	185	60.76	32.01	85.05	a	29
15	186	60.29	29.45	81.10	a	26
16	187	50.58	28.13	73.53	a	20
17	188	49.18	25.46	66.97	a	17
18	189	61.60	28.62	71.00	a	22
19	191	66.82	34.88	97.00	b	37
20	193	71.83	35.40	89.82	b	36
21	194	81.00	42.00	93.78	c	45
22	195	73.40	34.04	90.72	b	36
23	196	67.37	27.65	93.74	a	30
24	197	79.32	47.32	91.88	c	44
25	198	79.47	45.51	91.60	c	44
26	199	50.65	26.40	84.08	a	23
27	200	56.64	24.01	78.91	a	22
28	201	67.07	39.34	92.39	b	38
29	202	52.06	29.99	79.28	a	24
30	203	75.58	36.58	83.76	b	36
31	204	58.85	32.91	81.54	a	28
32	205	67.21	35.99	87.10	b	34
33	206	71.27	42.14	91.66	c	41
34	208	53.49	31.27	94.14	a	29
35	209	52.61	27.80	92.70	a	27

TABLE 3 (*Contd.*)

Serial No.	Sample No.	Liquid limit per cent	Plasticity index	Fraction passing 200 sieve per cent	Classification group	Group index
36	210	73.96	36.06	91.49	b	37
37	211	53.84	25.27	94.10	a	26
38	212	49.33	26.38	97.42	a	25
39	213	58.52	27.12	94.30	a	29
40	214	58.74	31.03	92.60	a	30
41	215	56.86	31.16	89.20	a	29
42	216	66.03	34.76	90.36	b	34
43	217	59.44	32.34	86.91	a	29
44	218	58.42	32.01	85.00	a	30
45	219	60.16	32.47	86.76	a	30
46	222	72.82	37.16	91.91	b	38
47	223	83.81	43.71	93.01	c	46
48	224	71.18	37.16	82.38	b	34
49	227	64.19	38.34	86.04	b	34
50	228	67.91	32.10	95.00	b	35
51	229	68.87	41.25	83.54	b	37
52	230	72.19	37.32	84.95	b	36
53	231	55.62	27.48	79.80	a	23
54	232	57.43	26.25	96.90	a	28
55	233	76.80	44.54	87.13	c	42
56	234	57.30	31.40	91.63	a	30
57	235	62.52	30.42	91.13	b	31
58	236	88.14	48.14	89.16	c	45
59	237	93.38	46.72	91.12	c	46
60	238	73.05	42.42	84.99	b	39
61	239	0.88	42.69	81.07	b	37
62	240	60.57	29.97	79.53	a	26
63	242	59.09	30.59	92.25	a	30
64	243	60.05	33.29	83.04	a	29
65	244	67.68	39.97	84.47	b	36
66	245	77.55	45.02	97.33	c	46
67	246	78.27	43.76	94.45	c	45
68	247	78.68	50.63	93.20	c	45

OBITUARY

We record with deep regret the death of Shri EVAGORAS POLYDOROS NICOLAIDES, B.Sc., Dipl. C.E. (E.C.P.), M.Soc. C.E. (France), M.I. Struct. E. (Lond.), M.I.E. (Ind.), who died suddenly of a heart attack at Lausanne, Switzerland, on 18th October, 1958 whilst on return to India from a business tour in Europe.

Shri. Nicolaides was born in 1907 and had his early education in Greece. He had his Engineering education at the famous Ecole Polytechnique in Paris, from where he graduated in Civil Engineering in 1929 after a brilliant academic career.

In 1930 he joined the Athens organisation of Mr. John C. Gammon, with whom he spent the whole of his working life in the closest friendship and collaboration. In 1931 he was transferred to the Bombay Office of J. C. Gammon Ltd. (now Gammon India Private Ltd.) as Engineer-Designer. From then on he went from strength to strength in his profession of Civil Engineering, and rose to be Chief Engineer of his Company in 1950 and General Manager and Deputy Managing Director in 1952, which post he held at the time of his death.

His eminence in the design and construction of all concrete structural work was recognized throughout India and his collaboration was sought on most of the technical committees appointed by the different Engineering Institutions in the country. He was a member of the Bridges Committee of the Indian Roads Congress, a member of the Prestressed Concrete Development Group and served on the Code Making Committees appointed by the Indian Standards Institution and the Indian Roads Congress. He represented India in a number of International Congresses held on subjects connected with his work and his contributions were considered to be most valuable.

Shri. Nicolaides was a member of the Society of Civil Engineers (France), Institution of Structural Engineers (Lond.) and the Institution of Engineers (India). He was the Author of a number of technical Papers and was awarded the Mitchell Medal for his Paper on 'Impact in R.C.C. Bridges as compared to Steel Bridges' contributed to the Indian Roads Congress in the year 1940.