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MADRAS HIGHWAYS

THE JOURNAL

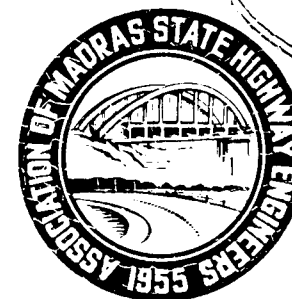
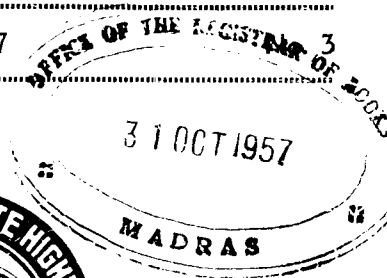
OF

THE ASSOCIATION OF
MADRAS STATE HIGHWAY ENGINEERS

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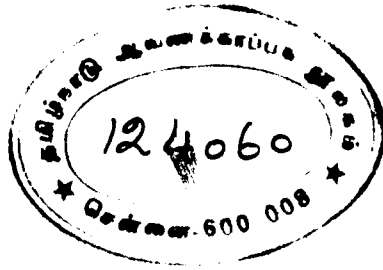
July 1957

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EDITOR



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The opening of the bridge across River Coleroon last month sets a land-mark in the history of development of Prestressed Concrete in our State. It was only a couple of years back the construction of the first Prestressed Concrete road bridge in this State viz. the Palar Bridge was completed and therefore we may justly feel proud that another major engineering feat has now been accomplished. While Palar Bridge represented a major Prestressed Concrete work on Magnel-Blaton System, the Coleroon Bridge is constructed on Freyssinet System.

During the past few decades technical science has widely expanded the frontiers of knowledge in the field of Prestressed Concrete from a simple idea of preventing crack formation in concrete and to make more efficient use of steel or concrete the new technical field has grown up. Prestressed Concrete has caused a revolution in the technique of Civil Engineering and particularly in Bridge Engineering. There is probably no structural problem to which prestressing cannot provide a solution—and often a somewhat revolutionary one but this is not to say that it will necessarily replace all forms of conventional construction: the introduction of steel did not do away with timber as a structural material. Therefore though Prestressed Concrete should not be expected to replace other forms of construction like that in steel and in Reinforced Concrete it has a very big role to play as a construction material.

The superiority of Prestressed Concrete is assured by the high quality of concrete and steel demanded in its manufacture; concrete of indifferent quality cannot be used in Prestressed Concrete work for concrete is loaded to 15 to 20 per cent higher than is it ever subjected to in service. Speed of construction is achieved by considerable use of precasting. But it would not be correct to claim that Prestressed Concrete must always be necessarily cheaper in first cost than other forms of construction. Under certain circumstances it certainly will be, this aspect

depending upon the type of structure, relative prices of various structural material available, basic conception of design etc. Popular conception of an economical structure is one which costs less to build than any other. This is fallacious; surely a superior article for the same price is more truly economical. Comparisons of initial costs are therefore not always the correct procedure, for economy of any structure is also intimately connected with its durability, with the cost of its future maintenance and with the speed with which it may be brought to service.

With bridge works, the advantages of Prestressed Concrete apart from the usual economy in time of construction, depends to a large extent upon the location and nature of the site and the relative costs of materials. For comparison of short span Reinforced Concrete and Prestressed Concrete proposals, relative costs of super structure is often taken as Criterion. This criterion is erroneously evolved by consideration of direct substitution of one type of deck by the other. Proper procedure should be to re-design the complete structure—for with Prestressed Concrete construction it is frequently possible to achieve economies by a re-arrangement of the substructure as well.

From the view point of national resources and in the present context of large scale construction activities, a prestressed structure is undoubtedly economical for it shows considerable saving in cement, aggregates and steel; and further more the construction work may be completed in considerably less time thus conserving labour also.

Granular Soil Stabilization

Contributed from Highways Research Station

(Contd.—from page 60)

4. Methods of Testing

Tests that are considered essential for the design and construction of stabilised soil roads are sieve analysis, liquid limit, plastic limit, standard compaction test and a method of determining density in the field. The first three of these help in understanding the general suitability of soils for stabilised soil work, the fourth in designing the best possible soil mix from given soils, and the fifth in controlling compaction in the field. The North Dakota Cone Penetrometer provides a very simple method of determining the relative bearing values of soils. It is easily used in estimating the relative merits of different mixes in conjunction with the standard compaction test and as a rough and ready method of compaction control whereby the actual number density determination in the field can be minimised. The use of the Cone Penetrometer for these purposes is explained in Chapter 3. All these tests are briefly described in the present chapter. At the end of each test suitable forms for recording the data and calculations are given so as to help the reader in getting a clear conception of the methods of testing. A section dealing with soil surveys is included in view of the undoubted importance of soil survey to highway engineering in general and soil stabilisation in particular.

4-1. Soil Surveys (S-01)

1. Purpose

The purpose of soil survey is to give the highway engineer relevant information as to soil and ground water conditions existing on road sites. The information is obtained by examination of the soil by hand boring methods and is used in the preparation of soil profiles showing the significant soil layers

or horizons. From such profiles and from laboratory investigations deductions can be made concerning :

- (a) Suitability of proposed location.
- (b) Selection of material for embankments and subgrade treatment.
- (c) Determination of cross sections for cuts and fills.
- (d) Requirements for subsoil and surface drainage.
- (e) Need for treatment of subgrades.
- (f) Type and thickness of base courses and surfacings.
- (g) Use of local material for soil stabilisation.
- (h) Pavement performance of road constructed over different soil conditions to develop general design formulae.

2. Equipment

The amount and type of equipment required for making a survey depends on the nature of the terrain and the particular purpose in view. The following list is considered fairly adequate under normal conditions but may be increased or reduced to suit actual conditions.

List of Equipment

One post hole auger with extensions.
 One gravel auger with extensions.
 One twist auger with extensions.
 Two small pipe wrenches.
 One light pickaxe.
 One shovel.
 One spade.
 Sample bags and twine.
 Tie on tags.
 Theodolite.
 Engineers level.
 Hand level.
 Levelling staff.
 One 100 feet steel tape or chain.
 Surveying arrows.

Ranging rods.
 Supply of wooden pegs.
 Sledge hammer.
 Note-books, boring charts and pencils.

Soil surveys are best done in conjunction with preliminary or location surveys so that the plans and profiles may be used readily for recording the soil survey data. If this is done the above list may be considerably reduced. It will facilitate field work greatly if a light motor van is available for carrying the soil survey party and equipment.

3. Preliminary Study of Soil

(a) *Geological Background*.—From the ground surface to a depth of about 6 ft., and exceptionally to a greater depth, the physical properties of the soil are influenced by seasonal changes of moisture and temperature and by such biological agents as roots, worms, and bacteria. The upper part of this region is known as A-horizon. It is subject primarily to the mechanical effects of weathering and to loss of some constituents due to leaching. The lower part is referred to as the B-horizon, where part of the substances washed out of the A-horizon are precipitated and accumulated. The Highway Engineer is more concerned with the A and B-horizons than with the underlying parent material.

Most soils were formed by geological processes that changed at random in space and time. Because of the decisive influence of geological factors on the sequence, shape, and continuity of the soil strata, the first step in any subsoil explanation should always be an investigation of the general geological character of the site. The more clearly the geology of the site is understood, the more efficiently can the program for soil survey be laid out. Valuable information in this direction may be obtained from a study of available geologic maps, topographic maps and agricultural soil maps. If no specific information regarding the geology of the site is available, the engineer must rely on his own capacity for geological observation and interpretation. In this connection the following brief description of the soils of Madras State will be of some use.

(b) *Typical Soils of Madras State.*—By and large the soils of Madras State are residual soils, Article 101, except in the coastal and deltaic regions where transported soils predominate. The peculiar monsoon conditions of India giving rise to alternating humidity and desiccation, had the effect of creating soils of a large variety not only in their texture but also in their mineralogical composition according to the nature of the parent rock from which the soils originated. Over the large areas of metamorphic rocks, the disintegration of the gneisses and schists has yielded a shallow sandy or stony soil; whereas that due to the decomposition of the basalts is a highly argillaceous, dark, loamy soil. All these soils exhibit a great deal of variation in depth, consistency and colour.

The soils of Madras State composite are broadly divided into five types.

Alluvial soils.

Black Cotton soils.

Red soils.

Laterite soils.

Sandy soils of coastal strips.

Alluvial Soils.—The alluvial soils in both coastal and deltaic areas are soils transported by rivers and so bear no relation to the rock on which they lie. They are found in the deltaic tracts of Krishna, Godavari and portions of South Arcot and Tirunelveli. Alluvial profiles are characterised by alternate layers of silt and clay separated from each other by layers of coarse sediments like sand.

Black Cotton Soils.—The Black Cotton soils are derived from two types of rocks: The Deccan and Rajmahal trap, and ferruginous gneiss and schists occurring under semi arid conditions. The former sometimes attain considerable depth whereas the latter are shallow. There is frequently no change in colour for 6 to 10 ft. depth. They are somewhat sandy on the slope and upland regions, and darker, deeper and richer in clay between the hills and plains, being constantly enriched by additions washed down from the hills. In all black cotton areas and

particularly those derived from ferro-magnesian schists, there is generally a layer rich in kankar nodules formed by the segregation of calcium carbonate at some depth below the surface and above the parent rock.

The Black Cotton soils in general have a higher content of finer fractions and may be classed as clay soils ranging from heavy clay to clay loam. As a result of having a high clay content they have a high liquid limit and they crack very deeply in summer owing to loss of moisture and shrinkage. The poor subsoil drainage and high surface drainage results in a certain amount of soil erosion.

The Black Cotton soils are found extensively in Bellary, Kurnool, Guntur, Krishna, Ramanathapuram and Tanjore. Guntur, Bellary, Kurnool and Krishna districts account for nearly half the black soil area in the State, the rest being distributed in other districts.

Red Soils.—The red soils occur in almost all districts and are to be found near the parent rocks. Generally these soils are not as deep as the black soils. The properties are exactly the opposite of the black soils. They are comparatively more open in texture being sandy to loamy, rarely clayey, and are easily drained.

Laterite Soil.—The laterite soils are formed in regions which are subjected to alternate heavy precipitation of over 100 inches of rainfall per annum and a dry hot period, for example the West Coast. If the parent rock contains iron compound in addition to the alumina the soil takes on a red colour. It has a vesicular structure and a freshly broken piece is mottled with various tints of brown, red and yellow and a considerable portion sometimes consists of white clay. Laterites are classified into high level or non-detrital and low level or detrital laterite. The two terms refer merely to the position occupied by the two varieties on the highlands or near the coast. The overlying surface soil is generally hard, well-drained and does not crack on drying.

Sandy Soils of Coastal Strips are composed of sediments that were eroded by waves or carried into the sea by rivers and transported and deposited by shore currents.

(c) *Boring Programme*.—After a careful study of whatever literature is available, a preliminary reconnaissance of the site is made by walking over the ground. All exposed soil profiles such as may be seen in quarries, pits and cuttings are examined, and the general topography and vegetation studied, such features as geological faults and unconformities looked for, and from the reconnaissance the interval between the borings decided on. These may be as much as 1000 feet in uniform ground, or as little as 50 ft. in rapidly changing ground.

4. Determination of Soil Profile

(a) *Setting Out*.—The positions for the bore holes are set out and marked with pegs, poles being used in long grass; each peg is numbered.

(b) *Borings*.—Normally, the depth of boring may be 4-5 ft. below existing ground level or proposed formation level, whichever is the lower. The depth bored can be less in valleys where filling is likely, and greater on hills where cuts may be required.

If a bed of poor soil such as soft clay is encountered, it should be penetrated completely and its boundaries located as exactly as possible. When rock is encountered it is obviously unnecessary to proceed further with the boring. Sites for borrow pits are also investigated by borings in similar way. After the boring is complete and the necessary sample taken, all holes should be filled in and the pegs collected for future use.

(c) *Simple Identification Tests for Soil*.—During sampling operations, the samples are roughly identified. With a little practice one can learn to identify approximately the various soils. The following are some of the simple field identification tests in use:

Cohesionless Soils.—A soil found in a loose state is sand or silt. If the soil is dry the amount of silt can be judged by

dropping the soil from a certain height and estimating the amount of dust created.

In the case of sand visual examination can help to estimate the size and shape of the grains (coarse, fine etc.) A magnifying glass may be of use.

Cohesive Soils.—If the soil is dry, crushing small lumps between the fingers helps to estimate cohesion.

If wet, rolling into threads gives an indication of its plasticity.

Silt can be distinguished from clay by rubbing a piece of wet soil between two fingers to feel the presence of gritty non-plastic material.

To assist in the selection of soils for use in embankments or for soil stabilization, a few simple tests may be performed in the field. If a sample of well graded moist soil material is squeezed in the hand, the following characteristics will be noted:

- (i) The soil is extremely gritty.
- (ii) It can be formed into definite shapes that retain their forms even when dried.
- (iii) If the clay alone adheres to the hands it will be enough to discolour them slightly.
- (iv) If more than enough soil to discolour the hands adheres to them, it will consist of both sand and clay, instead of clay alone.
- (v) When the moist sample is patted in the palm of the hand, it will compact into a dense cake that cannot be penetrated readily with the butt end of a pencil.
- (vi) If the moist sample is compacted into a container and allowed to dry, it will show little or no shrinkage.

(d) *Location of Water Table.*—If no trace of water be found in a boring, the hole can be filled immediately after sampling, but if water is found the hole should be left for 12 to 24 hours to enable the water to reach its final level, after which its depth below the surface is measured and recorded.

(e) *Records.*—Boring records are entered in Form ST-1.

(f) *Mapping the Soil Profile.*—The data obtained from boring records are plotted on the prepared profile map of the road for guidance in design. One method is to prepare a detailed soil profile showing the location of each significant soil layer observed during the examination of test borings made on the project. In another method on the location of the borings and the details of the soil layers as observed in them are shown. On the plan of the road the different soil types through which centre line cuts are also marked.

5. Selection of Soil Samples

A 10-lb. sample of soil is to be obtained from each soil layer on the basis of a study of boring records. A sufficient number of representative samples are taken so as to establish the range in test results for what appears to be the same soil layers. Each sample is placed in a canvas bag, marked with adequate identification, tied securely and sent to the laboratory.

6. Laboratory Test

The samples are tested in the laboratory. The usual tests are those required for identification and classification Article 102, and selected soils where they are to be used in subgrades or base course are also tested for compaction and bearing strength.

7. Analysis of Data

Recommendation for the engineering use of soil should be based on the soil properties and environmental characteristics as also the known behaviour of similar soils.

8. Subgrade Survey of an Existing Road

This is necessary for the following purposes :

- (a) Investigation of pavement failures.
- (b) Improvement projects.
- (c) Study of pavement behaviour.

Soils profiles are mapped in the same manner as suggested in para 4. The desired information may be obtained and recorded in the following manner :

(i) Scrape down the back slopes so that the original soil profile is exposed and plot the limits of the various layers on the prepared profile maps. The soil profile should be examined by auger borings to a depth of about 3 to 5 ft. below the formation level depending on the uniformity of the soil layers.

(ii) On the plan of the road, plot the limits of the various soil layers found directly under the surfacing material. For this purpose the information obtained by (i) above may be used if the soil layers are uniform. When the road is on embankment or in cutting through erratic soil pattern the limits are determined by auger borings in the shoulders.

(iii) Adequate samples are taken from each soil layer, para 5.

Analysis of Data.—These data together with the results of of laboratory tests are to be analysed and information concerning the following aspects obtained :

(i) The relation that exists between pavement condition the field characteristics of the soil, and the physical properties of the soil as determined in the laboratory.

(ii) The possible reasons for failure.

(iii) Possible corrective measures for the case under examination.

(iv) Preventive measures which may be applied in the future.

A representative test sample of the amount required to perform the desired test is then selected by the method of quartering or by means of a riffle box.

(*Quartering*.—The total amount of the soil sample is poured in the form of a conical heap and divided into four quarters. Two opposite quarters are rejected and removed out. The other two quarters are mixed together and the process repeated until the sample is reduced to the amount required).

(*b*) The portion of the air-dried sample selected for consistency tests is weighed and separated into two portions by means of a No. 40 sieve, if necessary after separating the larger sizes on a No. 10 sieve, and the fraction passing the No. 40 sieve is set aside for consistency tests.

(*c*) The portion of the air-dried soil selected for compaction test is separated by means of a No. 4 sieve. The fraction passing the No. 4 sieve is used for the compaction test.

4.3. Sieve Analysis of Soils

(S-03)

1. *Scope*.—This method describes a procedure for the quantitative determination of the distribution of particle sizes in soils upto No. 200 sieve.

2. *Apparatus*:

(*a*) *Sieves*.—A series of ASTM sieves of the following sizes : 1-inch, $\frac{3}{4}$ -inch, $\frac{3}{8}$ -inch, No. 4, No. 10, No. 40, No. 60, No. 100, and No. 200.

(*b*) *Drying Oven*.—Preferably with thermostatic control.

(*c*) *Balances*.—A balance to weigh upto 10 kg. and sensitive to 1 gm.; also a balance to weigh upto 500 gram and sensitive to 0.1 gram.

3. *Sample*.—See Article 4.2.

4. *Procedure*.

(*a*) *Drying*.—The sample is dried in stainless steel trays in an oven at a temperature of 100°-110° C.

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(*b*) *Breaking up Clods and Weighing*.—The dried sample may be rather lumpy, and, especially if clay is present it may be necessary to break up the lumps in a pestle and mortar; care should be taken not to crush the actual grains but only to break up any aggregation of grains. The sample is then weighed.

(*c*) *Wet Sieving for Removal of Clay*.—The sample is placed in a tray, covered with water and agitated to bring the silt and clay into suspension. Clean water is then poured on to the sample, agitated and decanted carefully on to a No. 200 sieve. This process is repeated until the wash water remains clear. Any material retained on the sieve is returned to the sample which is dried and weighed. The loss in weight is added to the material passing the No. 200 sieve in the following dry test.

(*d*) *Dry Sieving*.—Starting with the smallest sieve that will pass the whole of the sample, it is passed successively through the sieves (para 2.) The sieving operation is conducted by means of a lateral and vertical motion, accompanied by jarring so as to keep the sample moving continuously over the sieve. The portion of the sample retained on each sieve is weighed and the weight noted on Form ST-2. Before entering the weight of material passing No. 200 sieve the weight of material removed by wet sieving is added to it.

5. *Calculation*.—The data and calculations are entered in Form ST-2. The weight of the material retained on each sieve is calculated as a percentage of the total sample weight to the nearest 0.1 per cent. From these results the total percentage by weight passing each of the sieves is calculated to the nearest 1 percent. The grading is plotted in Chart ST-3. For specimen data and calculations see Form ST-2.

SIEVE ANALYSIS

(ST-2)

Name of work

Test No. File No.

Date Sheet No.

Wet Sieving

Wt. of dried sample before washing through No. 200 sieve = gm.

Wt. of dried sample after washing through No. 200 sieve = gm.

Wt. of material washed through No. 200 sieve = gm.

Dry Sieving

Sieve	Wt. retained gm.	Percent retained	Total % passing	Remarks
1 × Inch ...				
$\frac{1}{2}$ × Inch ...				
$\frac{3}{8}$ × Inch ...				
No. 4 ...				
No. 10 ...				
No. 40 ...				
No. 60 ...				
No. 100 ...				
No. 200 ...				
—200 dry ...				
—200 wet ...				
—200 total ...				
Total ...				

Description of Sample :

Remarks

4-4 Liquid Limit

(S-04)

1. *Definition.*—The liquid limit of a soil is that water content as determined in accordance with the following procedure at which the soil passes from a plastic to a liquid state.

Apparatus

2. (a) *Evaporating Dish.*—A porcelain evaporating dish about $4\frac{1}{2}$ in. in diameter.

(b) *Spatula.*—A spatula or pill knife having a blade about 3 in. in length and about $\frac{3}{4}$ in. in width.

(c) *Liquid Limit Device.*—This consists of a brass cup which is raised slowly by means of a cam to a height of 1 cm. above a hard rubber block from which point it falls freely on the block below, Figure 4-4-a. A standard grooving tool is employed for cutting a groove in the soil cake in the cup. The device is adjusted by means of a gauge attached to the grooving tool until the height of fall of the cup is exactly 1 cm.

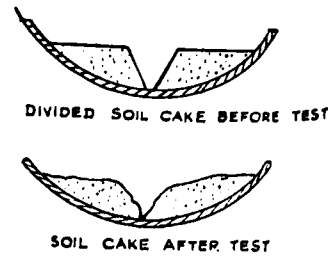
(d) *Containers.*—Suitable air-tight containers.

(e) *Balance.*—A balance sensitive to 0.1 gram.

3. *Sample.*—See Article 4-2.

4. *Procedure.*—About 100 gm. of the soil is mixed up thoroughly with distilled water to a suitable consistence with a spatula on a glass plate.

(a) Place approximately 50 to 80 gm. of the prepared sample in the cup of the liquid limit device above the spot where the cup rests on the base. With the spatula trim the soil to a depth of 1 cm. at the point of maximum depth and smoothen the surface. Divide the soil by a firm stroke of the grooving tool along a diametral line perpendicular to the axis of rotation of the cup so that a clear sharp groove of the proper dimensions is formed.



It is advisable to obtain the points corresponding to the greatest number of blows first, adding water by means of a pipette to obtain the consistency corresponding to the smaller number of blows. Additional material from the prepared sample should be added to replace the material taken from samples, if a full groove cannot be obtained.

(e) Weigh the container and the soil and weigh again after oven-drying at 110° C to constant weight and record the weights, on Form ST-4.

5. *Calculation.*—Compute the water contents in per cent dry weight from the ratio of the weight of the water removed by drying to the weight of the dry sample, Form ST-4.

Plot the flow curve on the semi-logarithmic chart ST-5 and determine the liquid limit as the water content corresponding to 25 blows.

4.5 Plastic Limit

(S.05)

1. *Definition.*—The plastic limit of a soil is the lowest water content determined in accordance with the following procedure at which the soil becomes plastic.

2. Apparatus.

- (a) Evaporating Dish, Article 4.4.
- (b) Spatula, Article 4.4.
- (c) Surface for Rolling.—A ground glass plate or rough textured paper.
- (d) Containers, Article 4.4.
- (e) Balance, Article 4.4.

3. *Sample.*—See Article 4.2.

4. Procedure.

WATER CONTENT AND CONSISTENCY TEST

(ST-4)

Name of work	File No.	Sheet No.
Test No.	Date	
Notes for Test: Water Content Test marked WC; Liquid Limit LL; Plastic Limit PL. Tare is the weight of container. No of blows refers to LL determination		
Water content = $\frac{\text{Weight of water}}{\text{Weight of dry soil}} \times 100$		
Sample No.		
Type of test		
Container No.		
No. of blows		
Weight of sample wet + tare, gm.		
Weight of sample dry + tare, gm.		
Weight of water, gm.		
Tare, gm.		
Weight of dry soil, gm.		
Percent water content		
Description of sample :		
Remarks :		

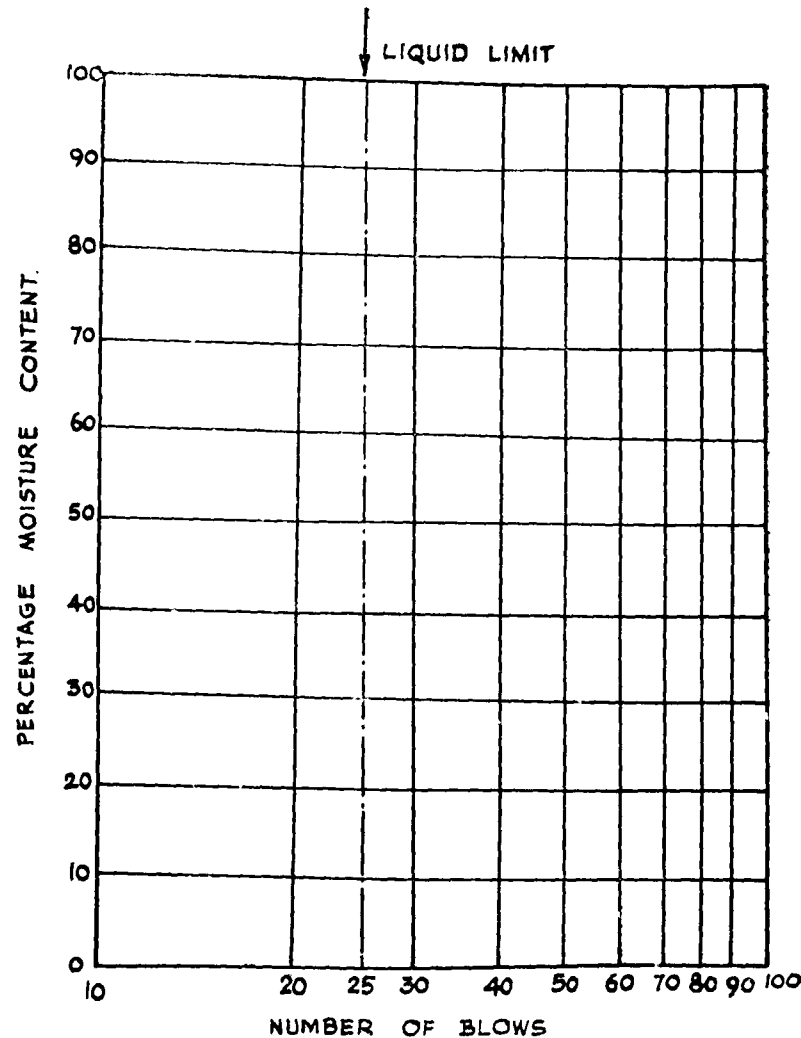
LIQUID LIMIT TEST

(ST-5)

Name of work

Test No. File No.

Date Sheet No.



Description of Sample :

Remarks :

(a) The soil is thoroughly mixed with distilled water until the mass becomes plastic enough to be easily shaped into a ball. The ball of soil is then rolled between the fingers and ground glass plate of rough-textured paper with just sufficient pressure to form the soil mass into a thread of uniform diameter.

(b) When the diameter of the thread becomes $\frac{1}{8}$ inch, the thread is broken into six or eight pieces and the pieces are squeezed and kneaded together into a uniform mass roughly ellipsoidal in shape which is then rerolled. This alternate rolling to a thread $\frac{1}{8}$ inch in diameter, gathering together, kneading and rerolling is continued until the thread crumbles under the slight pressure required for rolling.

(c) The portions of the crumbled soil are then placed in a suitable container and the moisture content of the soil determined, Article 4-4

5. *Calculations.*—The plastic limit is expressed as the moisture content in percent of the dry weight of the sample determined as above. The data and calculations are entered in Form ST-4.

4-6 Standard Compaction Test

(S-08)

1. *Scope.*—The object of this test is to determine the moisture content at which maximum density is obtained where the soil is compacted in a standard manner. The degree of compaction obtainable in the test is designed to be comparable with thorough compaction under field conditions. The test also gives the value of this maximum density and its relation to the dry density of the soil, *i.e.*, the bulk density of the soil after subtracting the weight of water present.

2. *Apparatus.*

(a) *Mould.*—A cylindrical brass mould capacity 1/30 cft, 4 in. diameter and 4-6 in high, fitted with detachable collar and base, Figure 4-6-a.

(b) *Rammer.*—Standard rammer, weight 5-5 lbs.

(c) *Straight-Edge*.—A steel straight-edge 12 inches in length.

(d) *Balances*.—A balance or scale of 25 lb. capacity sensitive to 0.01 lb. or equivalent metric balance and a 100 gm. capacity balance sensitive to 0.1 gm.

(e) *Drying oven*.—Preferably with thermostatic control.

(f) *Mixing Tools*.—Miscellaneous tools such as mixing pan, spoon, trowel, spatula, etc.

3. *Sample*.—See Article 4.2.

4. *Procedure*.

(a) The 6 lb. sample is thoroughly mixed with sufficient water to make it slightly damp so that when a portion is tightly squeezed in the palm of the hand, it will form a cast which will bear very careful handling without breaking. A compacted specimen is then formed by compacting the thoroughly mixed soil in the mould with collar attached, in three equal layers to give a total compacted depth of about 5 inches; each layer being uniformly compacted by 25 blows from the rammer dropping free from a height of 12 inches above the elevation of the soil. After the specimen is compacted, the collar is removed from the mould and the compacted soil carefully trimmed even with the top of the cylinder by means of the straight-edge. The weight of the soil contained in the mould is then determined.

(b) The specimen is extracted from the mould and a representative sample is taken from it for determination of moisture content.

(c) The remainder of the specimen is broken up, combined with the material left from moulding the specimen and steps (a), (b) are repeated with successive increments of water.

5. *Calculations*.—The wet density of the soil in the mould in pcf. is calculated from the formula.

$$D_w = 30 W$$

Where W is the weight of the wet soil in the trimmed mould in lb. If w is the moisture content in percent of the dry soil, the dry soil density in pcf. is

$$D_s = \frac{100 D_w}{(100 + w)}$$

The test data and calculations are entered in Form ST-6.

When a number of determinations have been made and the dry soil density is plotted against moisture content, Form ST-7, it is found that the curve connecting them has a peak corresponding to a maximum density. This is referred to as the Maximum Density and the corresponding moisture content as the Optimum Moisture content for this method of compaction.

4.7 Soil Density in the Field

(S-07)

1. *Scope*.—The object of this test is to measure the bulk density of soil or of base material *in situ* for the purpose of compaction control, Article 2.6.

2. *Apparatus*.—The apparatus consists of a 7 lb. glass jar of the type used by confectioners fitted with a conical head and pouring tap Figure 4.7-a. The tap is joined to a conical spout having a brass rim attached. The jar contains dry sand of known bulk density.

3. *Procedure*.—A tray with a 4 inch diameter circular hole in the centre is placed on the compacted surface and a neat hole 4 inches in diameter and 4 inches deep is excavated in the soil. The soil from this hole is collected and weighed and a representative sample is taken in an air tight tin for the determination of the moisture content in the field laboratory.

After being weighed, the glass jar and the pouring funnel are placed so that the pouring spout and base covers the hole in the soil. The tap is opened, and after a short time when no further movement of sand takes place in the jar the tap is turned off and the jar removed and weighed.

PROCTOR COMPACTION TEST (ST-6)

Name of work.....

Test No. File No.

Date Sheet No.

 D_w = Wet Density, pcf.= Wt. of wet sample in lb. $\times$ 30= Wt. of wet sample in gm. $\times$ 30
453.6 D_s = Dry Density, pcf.= $\frac{100 D_w}{100 + \text{water content}}$

Tare is the weight of container

Test No.							
Wt. of cylinder + sample wet, gm.							
Wt. of cylinder, gm.							
Wt. of wet sample gm.							
Density of wet sample, pcf.							

Water Content Determinations

Container No.							
Wt. of wet sample + tare, gm.							
Wt. of dry sample + tare, gm.							
Weight of water, gm.							
Tare, gm.							
Weight of sample dry, gm....							
Water content, percent							
Dry density of sample, pcf.							

Description of sample :

Remarks :

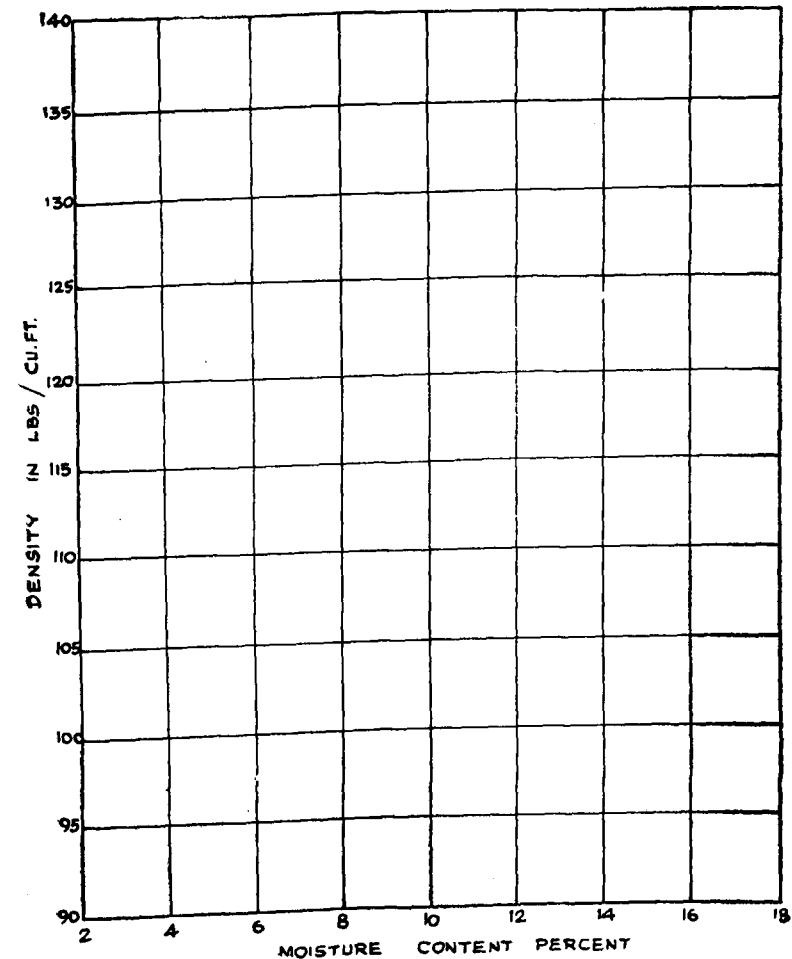
PROCTOR COMPACTION TEST (ST-7)

Name of work.....

Test No. File No.

Date Sheet No.

MOISTURE DENSITY CURVE



4. *Calculations.*—The difference in weight of the jar and attachment before and after the above operation gives the weight of the sand required to occupy the volume of the hole and the conical spout, the correction for the conical spout is determined separately by inverting the apparatus on a plane surface, say a sheet of glass. Since the bulk density of the sand is known, the volume of the hole in the soil can be found. From this volume, and the weight of soil removed, the wet density of the soil D_w is calculated and the density of the dry soil D_s is given by the formula :

$$D_s = \frac{100 D_w}{(100 + w)}$$

where w is the water content of the soil expressed as a percentage of the dry soil. The data and calculations are entered in Form ST-8.

5. *Bulk Density of Sand.*—The sand should be dry and the particles should be approximately of single size.

(a) Obtain a suitable container, clean and weigh it.

(b) To determine its capacity, fill with water and weigh again. The weight of the water in grams in the container will then be the capacity of the container in cc.

(c) Dry the container, fill with sand and weigh again.

The bulk density of the sand in gm. per cc.

$$= \frac{\text{Weight of sand}}{\text{Volume of sand} = \text{capacity of container}}$$

When the bulk density of the sand is expressed in gm. per cft, it may be more convenient to express this value as the volume in cubic feet per 100 gm. of sand, which equals

$$\frac{100}{1728 \times 2.54^3 \times \text{bulk density of sand}} \text{ cft.}$$

FIELD DENSITY TEST

(ST-8)

Name of work

Test No. File No.

Date Sheet No.

Density of sand used = gm. per cft.

 D_w = Wet density, pcf. w = Water content.

Weight of sand in cone = gm.

 D_s = Dry density, pcf. $D_s = \frac{100 D_w}{100 + w}$

Tare is the weight of container.

Sample No.					
Wt. of wet soil from hole, lb....					
Wt. of sand + bottle before pouring, lb. ...					
Wt. of sand + bottle after pouring, lb. ...					
Wt. of sand in hole + cone, lb..					
Wt. of sand in hole, lb.					
Volume of hole, cft.					
Density of wet sample, pcf. ...					

Water content Determination

Container No.					
Wt. of sample wet + tare, gm. .					
Wt. of sample dry + tare, gm. .					
Wt. of water, gm.					
Tare, gm.					
Wt. of dry soil, gm.					
Water content, per cent					
Density of dry sample, pcf. ...					

Description and Location of Samples :

- 1.
- 2.

4-9 Bearing Power of Soils

(S-09)

1. *Scope.*—The object of this test is to determine the bearing power of soils by means of the North Dakota Cone Penetrometer.

2. *Apparatus.*—A North Dakota Cone Penetrometer consisting of a vertical shaft fitted with a small loading platform at the top and a sharp steel cone at the bottom. A device is provided to measure the depth of penetration of the cone during test.

3. *Procedure.*—(a) The soil surface to be treated is levelled and the cone penetrometer is set in place and adjusted in such a way that the tip of the cone just penetrates the surface. The shaft is locked in this position by a set screw, loaded at the top and released slowly. The cone is then allowed to penetrate and settle down for one minute after which the shaft is locked again. The load is thus increased in successive stages to 80 lb. including the self weight of the shaft and the depth of penetration corresponding to this load is measured. The pressure in psi corresponding to this settlement is read off from the chart, ST-9, and this value is reported as the bearing power of the soil.

(b) For stabilised soil design purposes in the laboratory this test is performed on a slab 3 ft. $\times$ 3 ft. made of the trial mix.

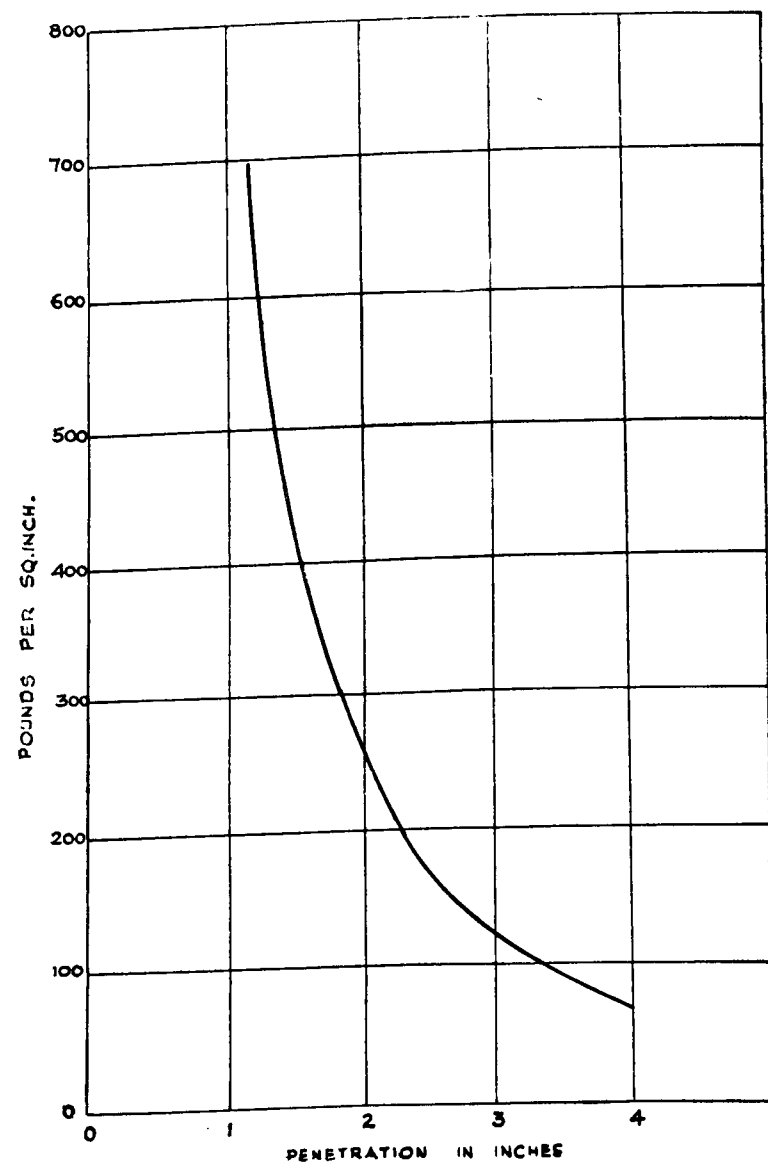
(c) *Bearing Power after 3-Day Soaking.*—A shallow pond 3 ft. $\times$ 3 ft. is constructed over the surface by means of a clay bund 4 to 5 inch high. The pond is kept filled with water for 3 days after which the water is let out and the bearing power determined on the firm surface which is exposed by scraping off the top $\frac{1}{2}$ inch or so of loose soil.

4. *Significance.*—The bearing power in psi determined by means of the cone penetrometer is measure of the ability of a soil to sustain highway loads.

The bearing power of a soil is adversely affected by the presence of excessive moisture. The bearing power determined

BEARING VALUE OF SOILS (ST-9)

WITH A NORTH DAKOTA CONE PENETROMETER
(80-Lb. Load)



after the 3-day soaking is expected to be the lowest value for the soil under the worst conditions of moisture that may be encountered. The 3-day soaking test is introduced as a step in the design of stabilised soil mixes, Article 3.3, so as to provide for the worst conditions of saturation. If the actual conditions are not likely to be so severe, the bearing power after the 3-day soaking may be increased by a suitable percentage depending on the anticipated degree of saturation and duration of wet conditions. This is largely a matter of judgement.

The 3 day soaking test is not applicable to sandy soils.

4.9. Moisture-Density Relations of Soil-Cement Mixtures

(S.10)

1. *Scope.*—This method of test is intended for determining the relationship between the moisture content of soil-cement mixtures and resulting densities (oven-dry weight per cubic foot) when the soil-cement mixture is compacted in the laboratory, before cement hydration.

2. *Apparatus.*—As per article 4.6.

3. *Sample.*—See article 4.2.

Procedure.—(a) The air-dry soil shall be first pulverized to pass a No. 4 (4760 micron) sieve so as to separate the soil particles without reducing the particle size.

(b) The required cement shall be added to the pulverized soil. The test shall be performed only on that portion of the soil-cement mixture which will pass a No. 4 (4760-micron) sieve. A quantity of water sufficient to produce slight cohesion shall then be thoroughly mixed with the soil-cement sample to permit ready compaction. The thoroughly mixed soil-cement sample shall be immediately compacted in the mould in three equal layers, to give a total compacted depth of about 5-in, each layer being compacted by 25 blows of the rammer dropping free from a height of 12-in. above the elevation of the soil. During compaction, the mould shall rest on a uniform, rigid foundation

weighing 200 lbs. or its equivalent. The blows shall be uniformly distributed over the surface of the layer being compacted. After compacting, the collar shall be removed and the top carefully trimmed to the exact height of the mould with a steel straight edge to produce a specimen approximately 4.6 in. in height and having a volume $1/30$ cu. ft.

(c) The weight of the compacted soil-cement mixture shall be determined, the material removed from the cylinder sliced vertically in the center and a 100-g. sample taken from the center, weighed immediately, dried in an oven at 110°C (230°F) for at least 12 hours or to constant weight. This procedure establishes the moisture density relation of the initial soil-cement.

(d) The soil-cement mixture shall be again finely pulverized, so as to separate the soil particles without reducing the particle size, to pass a No. 4 (4760-micron) sieve and a small increment of moisture carefully added and thoroughly mixed to insure uniform distribution. Then the moistened and pulverized material shall be compacted in the mould in three layers as described in paragraph (b), weighed, and a moisture determination made to establish the moisture density relation for a slightly moistened soil-cement mixture. This procedure shall be repeated with successive increments of water until the moisture content of the soil-cement mixture reaches a condition where it is difficult to pulverize the moistened materials, or it has a moisture content near the liquid limit.

4. *Calculations.*—See article 4.6.

4.10. Wetting-and-Drying Test of Compacted Soil-Cement Mixtures

(S.11)

1. *Scope.*—This method of test is intended for determining the soil-cement losses, moisture changes, and volume changes (swell and shrinkage) produced by repeated wetting and drying of compacted specimens of soil-cement mixtures of known composition and of known uniform density and moisture content.

2. *Apparatus.*—All items (a) to (f) as per article 4.6 and the following in addition.

(g) *Moist Room*.—A moist room capable of maintaining a temperature of 21 ± 1.7 C (70 ± 3 F.) and a relative humidity of not less than 90 per cent for seven-day storage of compacted specimens.

(h) *Water Bath*.—Suitable tank for submerging compacted specimens in water at about 21 C (70 F).

(i) *Wire Scratch Brush*.—A wire scratch brush made of 2-in. by 1/16-in. flat No. 26 gauge wire bristle mounted on a $7\frac{1}{2}$ -in. $\times$ $2\frac{1}{2}$ -in. hardwood block.

3. *Preparation of Material for Moulding Specimens*.—This test shall be performed on soil-cement mixtures compacted in the mould, before cement hydration, to maximum density at optimum moisture content and in the same manner of compaction as specified in article 4.9. Representative roadway soil shall be used except as modified in the following Paragraph (b).

(a) *For Soil Cement mixtures containing no material retained on the No. 4 Sieve*.—A sufficient quantity of each soil-cement mixture to be investigated shall be prepared at optimum moisture content to provide two compacted specimens approximately 4.6-in. in height and 4.0-in. in diameter.

(b) *For Soil Cement mixtures containing material retained on the No. 4 Sieve*.—When the soil-cement mixtures contain material retained on the No. 4 (4760-micron) sieve, the following procedures shall be followed which vary from that described in Paragraph (a): Since the specimens are 4-in. in diameter, it is necessary to prescribe the maximum size material to be used in the specimen and this size shall be set at $\frac{3}{4}$ -in. Compensation shall be made for any material larger than $\frac{3}{4}$ -in. by re-placing it with an equal dry weight of No. 4 sieve to $\frac{3}{4}$ -in. material of the same gradation as that contained in the original soil sample. A sufficient quantity of minus No. 4 sieve soil-cement material shall be prepared at the optimum moisture content to provide two compacted specimens 4.6 in height and 4.0-in. in diameter after the requisite amount of saturated, surface-moist, plus No. 4 sieve material, as defined above, has been added to the damp, minus No. 4 material.

4. *Moulding Specimens*.—(a) The prepared material shall be compacted in the mould in the same manner as that specified in article 4.9 with the further provision that before placing and compacting the next layer, smooth compaction planes produced during compaction of each layer shall be scarified to form grooves, at right angles to each other, approximately $\frac{1}{8}$ -in. in width and $\frac{1}{8}$ -in. in depth and approximately $\frac{1}{4}$ -in. apart. During compaction a representative soil-cement mixture moisture sample shall be taken from the batch, weighed, dried in an oven at 110 C (230 F) for at least 12 hr. or to constant weight. The compacted specimens shall be removed from the mould, weighed, and the density determined; the volume shall be determined by (1) averaging the diameters measured to the nearest 0.02 in. (9.05 cm.) at right angles to each other at the centre and the quarter points of the specimen and averaging the heights measured to the nearest 0.02 in. (0.05 cm.) at the quarter points on the circumference of the specimen, or (2) by immersion and displacement methods in a non-absorbent liquid such as mercury.

(b) One specimen shall be identified on a metal tag (or other suitable device) as No. 1 together with other needed identification marks and shall be used to obtain data on moisture and volume changes during the test. The other specimen shall be identified as No. 2 together with other needed identification marks and shall be used to obtain data on soil-cement losses during this test.

(c) After the required data have been obtained, the specimens shall be placed on suitable carriers in the moist room, and protected from free water for a period of seven days. The No. 1 specimens shall be weighed and measured daily to give data for calculating the moisture content and volume of the specimens each day.

5. *Procedure*.—(a) At the end of the storage in the moist room, the specimens shall be submerged in tap water at room temperature for a period of 5 hr. and removed. The No. 1 specimen (volume and moisture change specimen) shall be weighed and measured.

(b) Both specimens shall be placed in an oven at about 71 C (160 F) for 42 hr. and removed. Then, both specimens shall be weighed and No. 1 specimen measured. Specimen No. 2 (soil-cement loss specimen) shall then be given two firm strokes on all areas with a wire scratch brush to remove all material loosened during wetting and drying. The specimen shall again be weighed after brushing. The moisture content of the material brushed from the specimen shall be determined directly, but if this is not possible, it may be assumed to be equal to the moisture content of the No. 1 specimen. The oven-dry (110 C, 230 F) weight of material brushed from the specimen shall be calculated

(c) The procedure described in Paragraphs (a) and (b) constitute one cycle (48 hr.) of wetting and drying. The specimens shall then be again submerged in water and the procedure continued. The data collected will permit calculations of volume and moisture changes of specimen No. 1 and the soil-cement losses of specimen No. 2.

(d) The wetting and drying of the No. 1 specimens shall be continued for 12 cycles or discontinued prior to 12 cycles should the measurements become inaccurate due to distortion or soil-cement loss of the specimens. The wetting and drying of the No. 2 specimens shall be continued for 12 cycles, if possible.

6. *Calculation.*—The volume and moisture changes and the soil-cement losses of the specimens shall be calculated as follows :

(a) The difference between the volume of specimen No. 1 at the time of moulding and subsequent volumes shall be calculated as a percentage of the original volume.

(b) The moisture content of specimen No. 1 at the time of moulding and subsequent moisture contents shall be calculated as a percentage of the original oven-dry weight of the specimen.

(c) The soil-cement loss of specimen No. 2 shall be calculated as a percentage of the original oven-dry weight of the specimen.

Some Reflection on the Practice of Prestressing

BY

P. G. NAMBIAR, B.E., Divl. Engr. (H) Coleroon Bridge.

The Secretary suggested that I should write on some aspect of Coleroon Bridge Work which would be of some practical value to brother engineers. In this brief article I propose to deal with certain aspects of prestressed concrete construction which have constantly engaged my thoughts and attention at Coleroon on and which, if neglected, might have caused serious trouble during prestressing.

Construction Risks

The widespread use of reinforced concrete for bridge construction in our part of the country has brought about the virtual elimination of heavy lifting and carrying and the use of complicated construction machinery usually associated with steel bridges. With this the risks to workmen inherent in such operations have also been banished for good.

When we step from reinforced concrete to prestressed concrete, although the basic material is the same, the character of construction from the point of view of personnel safety undergoes a drastic change. With its preference for prefabrication and hoisting we go back to most of the risks of heavy steel construction, and to this some more are added thanks to the very concept of prestressing, involving as it does the imposition of permanent forces in the mass of the concrete.

Prestressing can turn Lethal

In the Freyssinet system which was employed at Coleroon the prestressing tendons were cables made up of 12 wires of 7 mm. diameter. In the process of prestressing, the wires are secured on the cuff of the jacks by means of simple wedges. In the final stage of stressing a cable the force in the cable in the vicinity of the jack

can be anything upto 60 tons, depending on the length and curvature of the cable and the kind of sheathing employed. Should the wedges become unseated in that state the mutual catapulting action between the jack and the cable released from grip while under great stress can be terrific. If all or almost all of the six wedges on a jack fly off simultaneously the catapulting force will be almost equal to the force in the cable at the moment of the release. A standard jack weighs 187 lbs. (Model 1951) and if it is hurled with a force of 60 tons as result of all the six wedges flying off at the same time it becomes a missile capable of spreading death and destruction along its trajectory.

Keep Clear of Beam Ends

I have heard of the occurrence on accident of this kind in England when a man was killed outright on the spot. No such unhappy thing has happened at Coleroon, but I mention the instance only to emphasise the importance of keeping the ends of a beam being stressed clear of work people for about 100 ft. either end. The jack must be slung from a well secured crane and the arrangement must be good enough to prevent the jack from taking to flight in the event of wedges giving way. Had it not been for these precautions continually and rigidly enforced there might have been a serious accident at Coleroon also, not with a flying jack, but with its complement, the extruded cable, which in this case came out at the other end for a length of 30 ft. in an instant, and could have penetrated anything in its path. A cold shiver runs down my spine whenever this incident comes to my mind.

Why not Prevent

The reader may well ask whether this unseating of wedges during prestressing cannot be prevented. The answer is that it can be, provided the condition of the wedges remains as Eugene Freyssinet probably intended and the weather also co-operates by remaining sufficiently free from humidity. Over the latter condition we have no control, and at Coleroon we had some bitter experiences of the effect of atmospheric moisture on the instability of wedges. When we first started the prestressing of the decking slab in October 1956 almost every morning we had trouble with the wedges flying off even though they were brand new and every conceivable

precaution was taken. If the weather was bright, after about 10 a.m. the trouble usually stopped. That as a very wet season with the air very humid in the mornings and further the actual conditions were much more hostile since we were operating right over flowing water. Our only solution to this difficulty lay in starting the prestressing operations only about 10 a.m. keeping the prestressing squad engaged till then in cleaning of cones and cables and other preparatory works for the day.

Although the wedges are made of special steel and case-hardened, being subjected to heavy concentration of stress, they wear out very fast. The V-shaped groove on either side in which the wires are gripped against the cuff of the jack wear down to a crescent shape after about a dozen applications, and then it is time for the wedges to be tempered, sharpened and case-hardened again by quenching the working edges alone in wax, which has been found to be a very good medium for this. All this requires some delicate handling in the hands of a really skilled workman. On the average a set of good wedges may be reconditioned some three or four time; after that they become too small and unfit for further use.

Some Practical Hints

There are some simple precautions which have been found from experience at Coleroon to go a long way in keeping prestressing operations free from trouble and ensuring good results.

1. A few hours after concreting, the cables must be moved back and forth within the sheaths by hammering first from one end, and then from the other, in order to prevent any blocking that may develop due to leakage of mortar through undetected rupture in the sheaths.

2. Mortar that may have found its way into the anchorage cone should be raked out and washed clean with a jet of water which is conveniently drawn from the pump set attached to each jack.

3. Before stressing a cable the inside of the anchorage cone should be thoroughly cleaned of all dirt, accretions and rust, and the clean metal of the reinforcing helix exposed.

4. The anchorage zone of the cables, that is about 4 ft. of their length at each end, should like wise be cleaned till the metal is smooth and clean. The grip of the wires in the jack is at its best only when the wires are in this condition.

Conditions 3 and 4 are essential for the perfect locking of the anchorage brought about by the forcing in of the male cone. It is incorrect to leave any rust on the wires with the hope of developing a better grip between cable and jack or cable and anchorage elements. The results will be contrary to expectations.

5. In stressing of short cables such as occurring in decking slabs, for the sake of minimising loss of cable stress due to locking of anchorage by means of the jack, it is necessary to perform the jacking from one end only while the other end is partially locked by the male cone pushed in manually. The success of this method depends upon the automatic completion of this locking while the cable is stressed from the jacking end. This involves an inward movement of the cone and cable on the offside as the jack pressure is raised. This movement must be smooth and uniform in all the wires. Otherwise the wedges on the jack are liable to fly off. In order to ensure this smooth and uniform movement conditions 3 and 4 above must be rigidly enforced, and further, the wires projecting beyond the face of the anchorage on the offside must be collected together and inserted securely in a 1-ft. length of 1½-inch pipe.

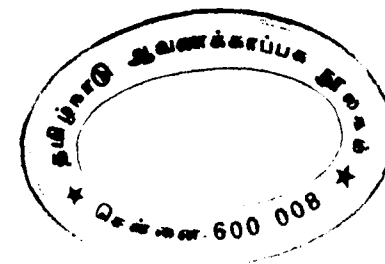
6. As regards the condition of the wedges, besides what has been mentioned earlier, the working edges of a wedge that is the V-grooves, should also be clean, smooth and dry. While the anchorage and the wires can be cleaned with emery paper, the wedges should only be cleaned with cloth since the grinding effect of emery may spoil the working edge. To completely remove the final and not so obvious traces of oil, grease and moisture the edge may be rubbed with a piece of chalk and afterwards cleaned with dry cloth. It was found worthwhile to repeat this operation every time.

Minimise Friction Losses

The designer stipulates the cable stresses to be developed at critical sections. In the field these stresses are generated by the external application of the jack at the extremities of the cables. Because of friction between cable and sheath these end stresses have to be higher than the design stresses, and as friction depends mainly on curvature and length of the cable, in the case of long cables or those with sharp curvature, the end stresses often approach values as high 80 tsi, which is very near the "elastic limit" or 0.1 proof stress for 7 mm. wires. There is nothing wrong in this, rather it is advantageous in prestressed concrete to work to high stresses since the safety of the structure against fatigue is thereby increased. For this reason and for the sake of economy the steel stresses in any well-designed prestressed concrete bridge—Coleroon for instance—must be as high as practicable, and there will not be much room to admit un-accounted friction, *i.e.*, that which cannot be accounted for by the cable profile.

Profile Must Stay Put

Most of this unaccounted friction which reveals during stressing arises from wobble of tendons between hangers. Since a cable naturally takes a parabolic shape between supports, it is very easy to secure a cable profile with the minimum of supports or hangers. But during concreting, under the weight and flow of concrete, this nice-looking profile is disturbed, and what was probably a smooth parabola is cut up into a series of undulations. The only way to prevent this lies in providing hangers or supports at sufficiently close intervals; for 7 mm. cables experience shows 5-6 ft. as the desirable spacing. The important point to remember is that we should not be satisfied with a profile that looks correct, but must ensure that it stays put during concreting. The drag on cables can be very heavy when stiff mixes are adopted and powerful external form vibrators are used for compaction.



MICRO-CLIMATE

Like the ancients we shall realize that animals birds and insects use the micro-climates of their environment to their own advantage and can teach us a great deal, for though man's interest in local climates has been casual in the past, the birds and beasts have always realized that they had a considerable influence on the comfort of their lives. Fifty years ago I learnt my first lessons on earth temperatures from a rabbits' air-conditioned burrow, and from the way a mole adapted himself to winter frost by making his runs under long grass where the ground was less likely to freeze. In summer when the nights are warm, cattle lie in the long grass where it is cool, but in spring and autumn when the nights are cold they seek out the warmer bare earth to lie on and are particularly fond of a macadam road. Motorists should know this well and drive very cautiously at night on unfenced roads across commons where cattle graze.

—(*From Climates in Miniature by T. Bedford Franklin*).

* * *

THE GREAT PYRAMID

X X X Without recalling here the blocks of stone weighing 1500 tons, transported by unknown means over enormous distances, or the rocking-stones, masses of rock weighing five hundred tons, which were never native to the soil upon which they now rest, and which date from the prehistoric era of the Atlanteans, it is an undoubted fact that the great pyramid of Cheops, for example, is a sort of stupendous hieroglyph, which, by its dimensions, its proportions, its internal arrangements, and its astronomical orientation, propounds a whole series of riddles of which only the most obvious have hitherto been deciphered. An occult tradition had always affirmed that this pyramid contained essential secrets, but only quite recently has any one begun to

discover them. Abbe Moreux, the learned director of the Bourges Observatory, giving a complete summary of the question in his *enigmes de la science*, shows us that the meridian of the pyramid—the line running north and south passing through its apex—is the ideal meridian; that is, it is that which crosses the greatest amount of land and the smallest amount of sea, and if we calculate exactly the area of habitable territories, it will be found to divide them into two strictly equal halves. On the other hand, if we multiply the height of the pyramid by one million, we obtain the distance from the earth to the sun, or 198,208,000 kilometers, which is, within about one million kilometers, the distance which modern science has finally adopted, after long research and dangerous expeditions to distant lands, and thanks to the progress of celestial photography.

The well-known astronomer Clark has calculated, from recent measurements, the polar radius of the earth. He makes it 6,356,521 meters. Now this is precisely the cubit of the pyramid-builders, or 0. 6,336,321 meters, multiplied by ten millions. Next, on dividing the side of the pyramid by the cubit used in its construction, we have the length of the sidereal year; that is, the time which the sun requires to return to the same point in the sky. Then, if we multiply the pyramid-builders' inch by one hundred millions, we shall obtain the distance which the earth travels in its orbit in one day of twenty-four hours, the approximation being closer than our modern measures—the yard or the meter—would permit of our making. Lastly, the entrance—passage of the pyramid pointed toward the pole star of the period; it must therefore have been orientated with reference to the precession of the equinoxes, according to which phenomenon the celestial pole returns, coinciding with the same stars, after the lapse of 25,796 years.

We see, then, that, as Abbe Moreaux tells us, "all these conquests of modern science are found in the Great Pyramid in the form of natural dimensions, measured, and always capable of measurement, needing only opportunity to shine forth in broad daylight with the metrical meaning contained in them.

It is impossible to attribute these extraordinary data to mere coincidence. They prove that the Egyptian priests, in geography,

mathematics, geometry, and astronomy, possessed knowledge that we are barely beginning to reconquer, and there is nothing to tell us that this enigmatic pyramid does not contain a host of other secrets which we have not yet discovered. But the strangest most disconcerting fact is that none of the innumerable heiroglyphs that have been deciphered, nothing, indeed, to be found in the literature of ancient Egypt, makes any allusion to this extraordinary knowledge. X X X

—(From 'The Great Secret' by Maurice Maeterlinck
Translated by Bernard Miall).

*

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*

"E PUR SI MUOVE"

And still it moves—the words of Galileo, murmured when the tortures of the Inquisition had driven him to recant the Truth he knew, apply in a new way to our world today. Sometimes, in the knowledge of all that has been discovered all that has been done to make life on the planet happier and more worthy. We may be tempted to settle down to enjoy our heritage. That would indeed be the betrayal of our trust. These men and women of the past have given everything—comfort, time, treasure, peace of mind and body bite itself—that we might live as we do. The challenge to each one of us is to carry on their work for the sake of future generation.

The adventurous human mind must not falter. Still must we question the old truths and work for the new ones. Still must we risk scorn, cynicism, neglect, loneliness, poverty; persecution if need be. We must shut our ears to the easy voice which tell us that human nature will never alter as an excuse for doing nothing to make life more worthy.

Thus will the course of history of mankind go onward and the world we know move into a new splendour of those who are yet to be.

—(From *ideas that moved the world* by Horace Shipp).

HIGHWAY PLANNING AND CONSTRUCTION IN FRANCE

The French highway system is one of the most closely knit in the world. There are about 48,000 miles of main or national highways, 144,000 miles of secondary highways, 180,000 miles of first and second class roads, or a total of 372,000 miles hard surfaced roads, without counting rural roads and the streets of towns and villages, serving the whole of country which covers 212,655 square miles.

The broad and well laid-out national highways follow straight lines in greater part and embody the concern with strategy and centralization which determined under the French kings and Napoleon the development of a dense network of roads.

The first problem in the improvement of the highway system made its appearance when the increasing speed of the cars raised clouds, little appreciated by those living adjacent to the highways and the drivers themselves. Plans for tarring roads were put under way beginning in 1920 and nearly all roads were tar-surfaced by 1930. By the time this had been accomplished the volume of traffic had appreciably increased in the meantime as shown by the rise in the number of cars from 230,000 in 1920 to 1,500,000 in 1930 and many of the roads, which were only 12 to 15 feet wide, had become insufficient.

At that time it was assumed that the normal width of a traffic lane was 9 feet and that the national highways therefore should be at least 18 feet wide. For certain very heavily travelled roads, widths of 27 and 36 feet were established. A width of 21 feet, corresponding to two lanes each 10-1/2 feet wide, was fixed for roads carrying very heavy traffic and plans for widening the roads were prepared.

Delayed initially by lack of funds, these plans were being carried out shortly before the war within the framework of a general program of public works. At that time, the general policy for the improvement of the highway system in France was established.

This came to be known as the "Policy of National Highway-Improvement" and was both cautious and bold. Whereas other countries had precipitated themselves into the construction of superhighways between their large centers, France restricted itself to improving existing highways but the plans for these improvements were at that time much more radical and farreaching than anything else that had been undertaken before in France. There was in this no intention of denying the value of superhighways as proved by the fact that the construction of the "Superhighway to the West" at Paris and the plans for the "Croix Rousse Tunnel" at Lyon were undertaken at that time. The improvement of the main highways, which were then called "the international highways", was further supplemented by a program for improving the roads within the areas of Paris and Lyon.

During the occupation, the "Policy of National Highway Improvement" was continued but at a very much slower rate within the "10 year Plan for Economic Development".

At that time, the basis for the improvement of the main highways were further defined and the concept of "basic speed" was developed, which determined the characteristics of a roadway except for its width. This speed is considered to be that at which a driver can safely travel on a certain type of road under normal traffic conditions. Three types of characteristics were finally adopted for the main highways and based on travel speeds of respectively 72, 60, and 40-48 miles per hour. In accordance with terrain features, each of the main highways was divided into certain sections each corresponding to a given basic speed, with the transition from one speed to another as gradual as possible.

With this, the specific factors for a planned and coordinated improvement of the main highways had been determined for the first time.

In 1952, appropriations for highway construction were created by the "Fonds d'Investissement Routier" and not until then could measures be taken to remedy a situation which was becoming critical. A five-year-plan was undertaken which included the following separate projects:

1. Continuation of improvements in main highways as projected by previous programs and brought up to date on the basis of the definitions decided upon in 1952;

2. Completion of the superhighways to the West from Paris and to the North from Marseille and Lyon and start of work on superhighways to the South from Lille and Paris and to the East from Marseille.

3. Development of the road network of the urban areas of Paris, Marseille, Lyon, and Saint-Etienne, together with the industrial centers of the North and the East of France.

4. Construction of the Month Blanc tunnel;

5. Elimination of level railroad crossings;

6. Reconstruction of main bridges inadequate for present-day needs;

7. Reconstruction of roads in war-damaged areas;

8. Creation of scenic highways principally in the Alps, the Pyrenees, and along the Mediterranean Coast.

9. Miscellaneous projects selected on a carefully determined basis of priority. This selection is based on traffic counts, the costs of construction, and the advantages to the highway user by the savings in operating expenses and the increased safety inherent in modern road construction.

French standards published in 1952 relative to the requirements for construction of main and other national highways have made some changes in preceding texts and follow the requirements for the construction of international traffic roads as agreed upon at the convention in Geneva in 1950. They provide for standard traffic

lanes of 10—1/2 feet and a width of 31—1/2 instead of 27 feet for three-lane roads. They also modify the standards of construction for roads with two separated lanes which have a capacity practically 4 times greater than that of the standard two-lane road whereas that of a three-lane road is only 1/1/2 times as great.

These new standards introduce the principle of stabilized shoulders flush with the pavement level, particularly where it is necessary to facilitate parking, and recommend, wherever possible, planting of the shoulders only on the far side of the drainage ditches. They also define and prescribe the conditions for providing and constructing bicycle paths and modify the rules for the construction of curbs.

The present policy of improving the main highways by widening, straightening, and eliminating danger points will be continued but it is obvious that, on heavily travelled roads where traffic already exceeds 2,000 vehicles per day in 1950, this will be a short term policy. Within the limits of the available right-of-way, it is always possible to increase traffic to a certain extent by widening the road but in most cases the additional width very soon becomes again inadequate. A theoretical solution to this is given by then constructing a second road at a certain distance from the first and instituting one-way traffic on both the old and the new road but actually the problem of local traffic becomes nearly insoluble with such a one-way system. At a certain stage, it therefore will be necessary to abandon the existing road completely to local traffic and to contemplate the construction of superhighways along the main long distance traffic arteries such as the one planned for many years between Paris and Lille.

In view of the actual uncertainty of any predictions for the future and the inadvisability for heavy capital investments in costly long range solutions which may remain unproductive for a long time, three main phases present themselves which should be undertaken now:

1. Improvement or reconstruction of highways to adapt them to the requirements of traffic anticipated for the next ten years and which should be carried out as soon as circumstances permit;

2. Changes or minor reconstruction possible immediately on the basis of existing funds or opportunities;

3. Long range construction for traffic needs which may be reasonably anticipated in the far future.

A concrete example is furnished by the Rhone Valley highway. The heavy traffic on this national highway between Lyon and Marseille, where actual count has determined a year-round daily average exceeding 6,000 vehicles, makes it appear that a superhighway would be the only solution on this stretch. Several possible new traces were examined and final conclusions showed that everywhere, except between Lyon and Valence, the superhighway would have to follow very nearly exactly the present road. This raised the problem of whether to continue with the construction of the by-passes already started on the national highway as well as with the widening of the present roadway to 27 feet, or whether to stop all work and wait for the construction of the future superhighway. Since the construction of a superhighway is a long term project, it was found absolutely necessary to continue with those by-passes which were both the most urgent and least expensive, and then to start superhighway in sections where it would eliminate other by-passes originally planned for the old road.

It was therefore decided to plan on construction of the urgent by-passes with two 31-1/2 foot lanes in view of their later incorporation into the future superhighway. However, for by-passes with a length of at least 3 miles, the principle of an immediate phase of construction of two 22-1/2 foot wide roadways—standard width of the future lane—was admitted since it was felt that the incorporation of such sections in the present highway would not present any danger and would facilitate overtaking of vehicles which often have to follow each other in convoy at the limit of capacity of the 27 foot sections.

However, for shorter by-passes which will be constructed with a 27-foot width, the transition to the future phase (two 21 foot lanes) will bring about the partial loss of 6 feet of roadway which will then be incorporated into the shoulders. It was also realized

that the future superhighway could be constructed not only in successive sections but also in successive phases or by constructing at the start only one roadway reserved for fast traffic.

This brief review of the great lines of highway planning in France leads automatically to an examination of the progress made in road-building by new methods based on increasing mechanization. One of the first new developments concerned the selection of high-output mechanical equipment to do away with costly labor in the placement of macadam. Such equipment was found in blade graders, bought out of Allied surplus war materials, one of which prepares enough work for three or four rollers operating simultaneously, with a saving in manual labor of 60%.

The grader was, however, not suitable for all sites so that attention turned to an increased use of spreaders, either of the trailer or automotice type. The latest models are capable of placing 600 tons of crushed rock per day with elements as large as 5 inches and over a width varying from 4-1/2 to 7-1/2 feet.

Progress has also been made in the design of spreaders, certain types of which spread the binder under low pressure nearly flush with the soil and to a maximum width of 10 1/2 feet with uniform edges. Some French manufacturers have designed vibratory spreaders which distribute not only gravel fines and coarse sand but also wet sand and adhesive materials with a uniformity never obtained before. The development of this equipment has brought about the design of loading equipment which is capable of a volume of 130 cubic yards per hour and can be easily moved such as bucket loaders.

Another important step forward has been the gradual substitution of pneumatic for smooth rollers. The latter have the disadvantage of cracking all but the very hardest gravel and even with the latter the percentage of cracked elements was not negligible. The result of this was premature ageing and weak points in the pavement. Reduction in weight of the rollers brought about no improvement in these disadvantages. Automotive pneumatic rollers give excellent results and have a much higher productivity than smooth rollers.

The construction of a roadway with an accurate trace and cross-section presupposes, independently of the thickness of suitable materials, very careful compaction and grading of the subgrade. Until not too long ago, natural settlement was still the only factor on which the engineer could really rely. The appearance of the sheep-foot roller, compacting relatively thin and successive layers, seemed the solution for reducing the period of natural settlement but its use required final surface compacting by means of smooth rollers. The advantage of compaction by pneumatic equipment were discovered after the substitution of rubber-tired equipment for the then currently used construction railways. Designers soon began to examine means of obtaining increasingly larger contact surfaces without any change in penetration. This led to the gigantic tires and heavy equipment of today but there is still a good deal of discussion going on in regard to the relative merits of heavy or light rubber tires and suitable tire pressures and the advantages of automotive or trailer equipment.

Vibratory compaction has been under experimentation for some years and is very suitable for coarse and fine sand and gravel, particularly when they contain a high percentage of water. Vibratory wheel loads of about 3-1/2 tons make it possible to compact layers with a thickness of nearly 3 feet and are more easily maneuvered than very heavy pneumatic rollers but the latter remain the proper equipment for the compaction of large areas and to show up any unequal consistency of the soil.

The development of the use of coated materials in road construction is one of the keystones of French road building methods. The following few data will permit an evaluation of the result obtained in this respect. During 1955, there were installed 1,500,000 tons of hot-laid bituminous pavement, over 500,000 tons of tar and cutback materials prepared in plants or fixed installations, and over 150,000 tons of cold-laid coated materials (in part utilized for road upkeep) or a total of over 2,300,000 tons. During 1944, the total of coated materials installed was only 1,500,000 tons, of which 600,000 tons were bituminous pavement.

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Another proof is furnished by the fact that the road building contractors during the last five years have quadrupled their equipment to reach a capacity of 3,000,000 tons in 1955. About one-half of this is constituted by units with a capacity of 100 - 120 tons per hour, one-sixth by units of 60 tons per hour, and the remainder divided among units of 20 to 40 tons per hour. Units of 100 - 120 tons per hour generally operate without screening and dosing equipment and with two finishing spreaders.

—(from *Highway Planning & Construction in France*
Published by *Alliance Francaise*.)

BITUMINOUS BASE COURSES

By

J. GREUTERT, (DIPL. ING. E.T.H.)

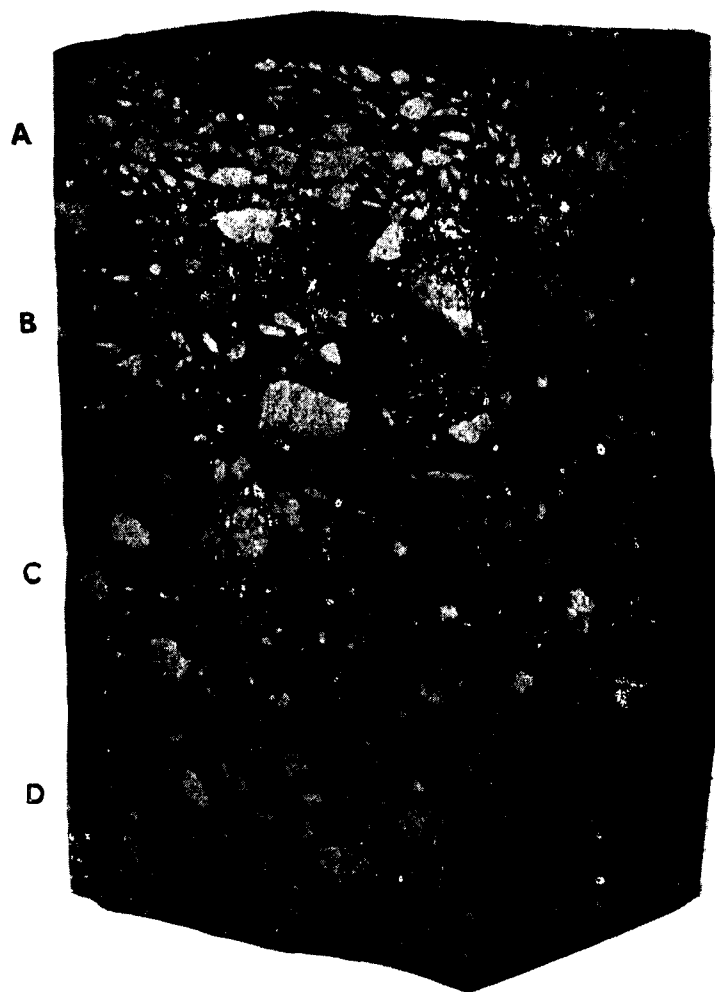
One of the functions of a road structure is so to distribute and reduce the stresses caused by traffic loads that the subgrade will not be permanently deformed. In order to be able to design a road structure for this purpose, a knowledge is necessary of the distribution of stresses when using materials of varying properties and thicknesses.

In recent years the Road Research Laboratory in Great Britain (1) have carried out investigations into stress distribution in the subgrade. It was found that the decrease in stress with increasing depth depends a great deal on the type of bituminous surfacing: thus it was shown that subgrade stresses were considerably less under dense hot-mixed asphaltic concrete than under bituminous macadam. Temperature, too, plays a part which is now under investigation.

Trials

Base course trials at Lahr in Germany (2) gave the opportunity of carrying out vibration measurements to determine the characteristics of various base courses. These field measurements showed surprising conformity with theoretical calculations (3) based on the relation between properties of materials, thickness of individual courses and surface deflection under load.

These trials and calculations led to the application, on a larger scale, of bituminous base courses in several countries.



Sample cut from a main road in Germany showing typical three-layer bituminous base course construction.

- A 3cm. wearing course (asphaltic concrete) on $\frac{1}{2}$ cm. binder course.
- B } Base Course: $3 \times 5\text{cm.} = 15\text{cm.}$ hot-mixed gravel sand with 5%
 C } filler and 5% 80/100 pen. bitumen.
 D }

Large-Scale Work

In Germany, the first large-scale sections were laid at Lauenburg (4) and Nievenheim (5), and at Augsburg (6). On main roads in Bavaria more than half a million square metres of bituminous bases were laid during 1956 and tenders have been called for a 15-cm. bituminous base course for 12 km. of the new autobahn Frankfurt-Nuremberg.

German experience has shown that a maximum particle size of 35-40 mm. is the largest which can be used economically with present mixing equipment. Nearly all bituminous base courses, therefore, use gravel having a maximum size of 35-40 mm. Local gravels are used and the sand content (smaller than 2 mm.) is usually about 20-30%. The void content varies from 15% to 18% and the necessary binder content from 4% to 5%, the binders used being 60/70, 80/100 and 180/200 pen. bitumen. Unfilled voids in the compacted bituminous base course layers vary from 3% to 8%.

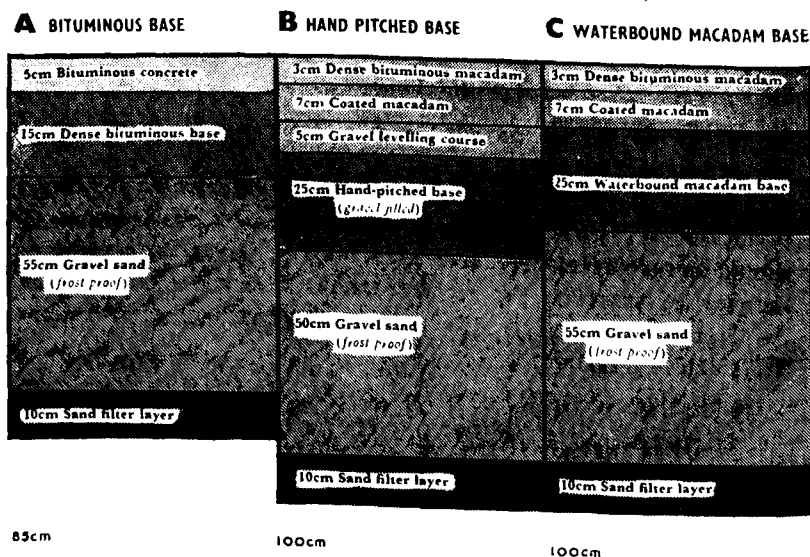
Whilst bituminous bases have gained prominence on the Continent due to the current large programme of new construction, it should be noted that they have been used in Great Britain for a number of years for the reconstruction of city streets and the re-alignment of existing roads, including strengthening, widening and superelevation work.

In Austria, plans for 1957 provide for bituminous base course work in Vorarlberg on the Main Road No. 1—Bregenz-Vienna—between Lauterach and Dornbirn and at Hohenems. Rhine gravel sand will be used after removing oversize material (> 35 mm.) This oversize material will be crushed and added to the sand/gravel mixture. The bituminous base course will be 15 cm. thick and laid in two courses by means of finishers. Considerable settlement is expected in this area (the subgrade is peat and loam) and it is therefore intended to surface the road with asphaltic concrete only after an interval of two or three years. In the meantime the base course will be given a temporary surfacing of a thin carpet or surface treatment.

Three Alternatives

Tenders for the work in Vorarlberg called for the three alternatives shown in the diagram.

The total required thickness of road structure by conventional design methods was calculated at 100 cm. The reasons for the reduced total thickness of the structure with a bituminous base (Alternative A) is that allowance has been made for the dense bituminous base course layers having a shear strength up to 2.5 times that of mechanically stabilised layers. This was confirmed in practice by the *Washo* road test (7), where increasing thickness of the bituminous surfacing from 5 cm. to 10 cm. indicated that a decrease in total road structure thickness would be permissible in the approximate ratio of 1 : 2. French results (8), too, showed similar ratios (1 : 2-2.5, in some cases even up to 1 : 4).



Special Features

Some of the specific advantages which appeal to road engineers using bituminous bases are :

1. Each layer of a bituminous base course can be opened to traffic as soon as it is completed. This cuts down traffic dislocation, allowing earlier use of the road than with other types of base course construction.

2. The spreading of consecutive courses by mechanical finishers should give an excellent final riding surface, as any irregularities in previous layers are diminished with each lift of bituminous base course material.
3. The greater degree of mechanisation in mixing and laying will lead to an overall lowering of construction costs.

When designing bituminous base courses, the following characteristics should be considered :

Density.—High density should be aimed at, with a voids content in the compacted layers 3% to 8%. This will ensure an impermeable base which will be unaffected by frost and also give maximum durability to the road structure.

Stability.—No permanent deformation must occur in the structure under static or dynamic loads, even at high summer temperatures.

Flexibility.—The base course must be sufficiently flexible to conform to any long-term settlement of the subgrade which may occur.

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—(Extracted from Shell Bitumen Review No. 1)

NEWS

It is expected the building of the Highways Research Station will be opened by Shri Jawaharlal Nehru, Prime Minister, by the end of October 1957. The Annual Session of the Association is proposed to be held in the Lecture Hall of the Highways Research Station soon after. Members will be intimated about the date. It is hoped all the members will participate and make the session a grand success.

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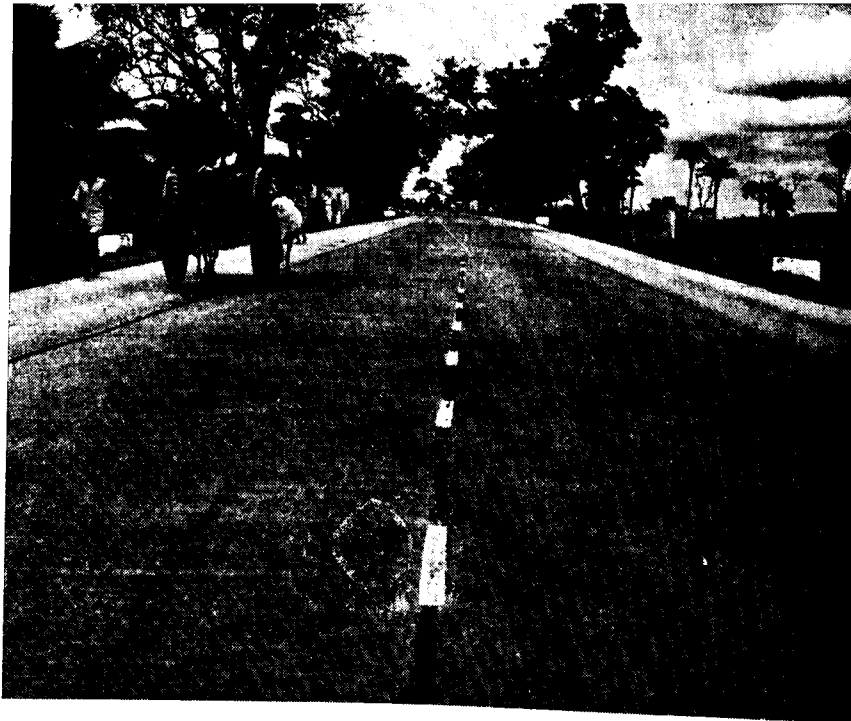


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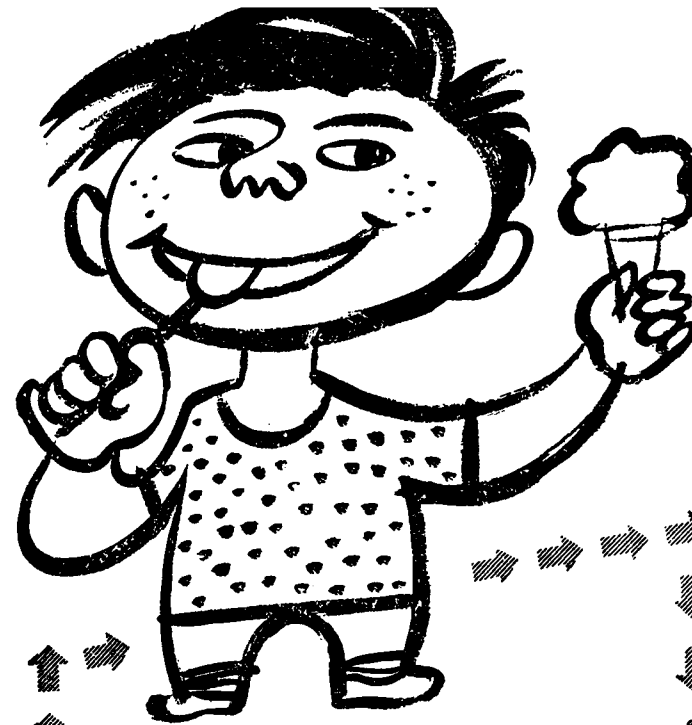
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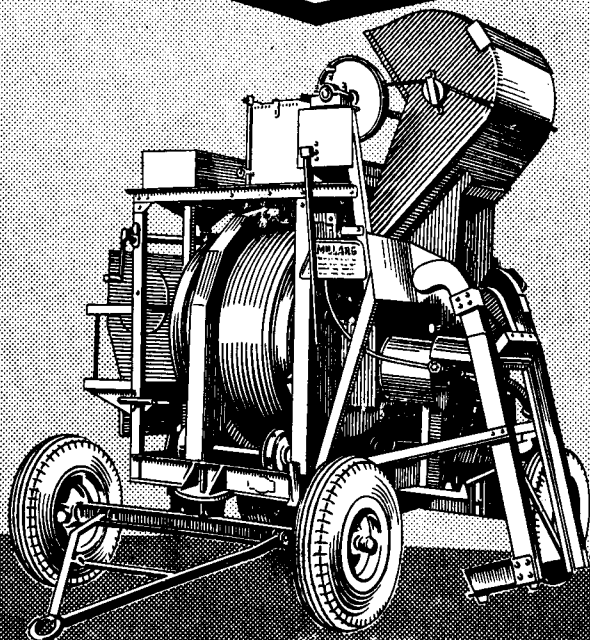
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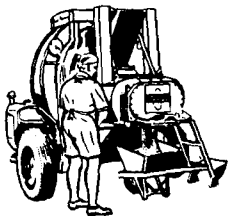
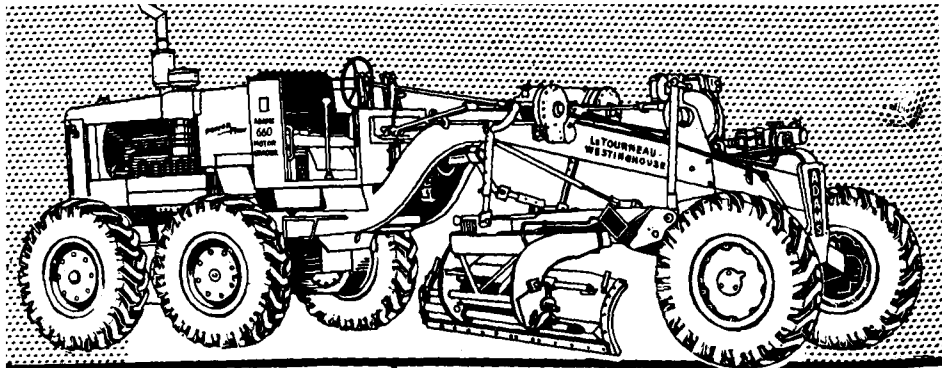
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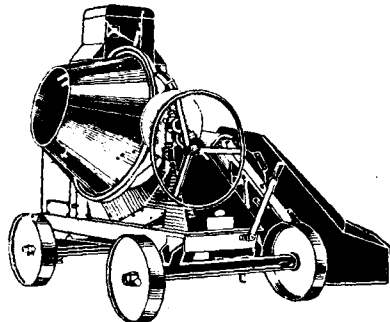
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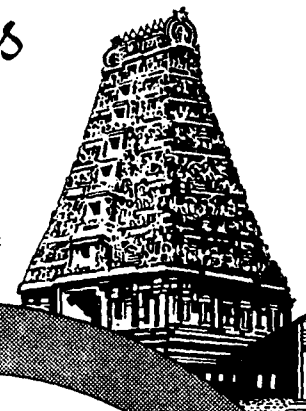
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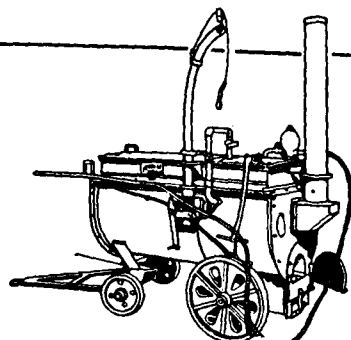
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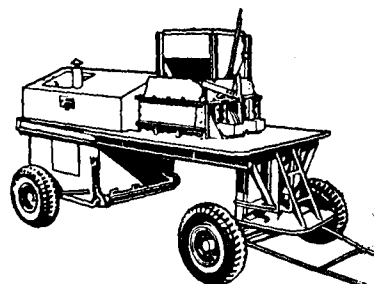
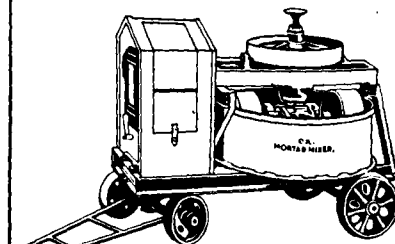


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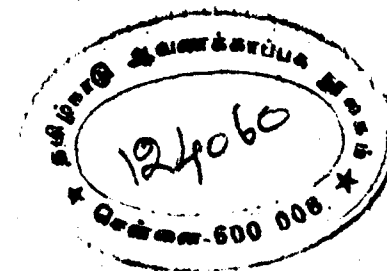
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He has achieved success who has lived well, laughed often and loved much; who has gained the respect of the intelligent men and the love of the little children; who has left the world better than he found it, whether by an improved poppy, a perfect poem or a rescued soul; who has never lacked appreciation of earth's beauty or failed to express it; who has always looked for the best in others and given the best he had, whose life was an inspiration, whose memory a benediction.

—Mrs. A. J. STANLEY.



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IMPORTANT

*Members are requested to notify
the Secretary*

AT ONCE

*any change of address or of
designation*