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CE 3206 (Sessional)

HIGHWAY MATERIAL TESTING

(LABORATORY MANUAL)

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Preface to the first edition

Different educational institutions in India have now developed their syllabii wherein testing of highway materials have been included. Hence an urgent need of a suitable laboratory manual has been felt. This book though particularly designed for engineering students, deals with the routine tests that the field engineers engaged in paving jobs may also need to carry out.

In preparing this manual, authors have extensively used the publication of ISI, BSI ASTM, and Asphalt Institute. The help received from these publications is therefore gratefully acknowledged here.

Authors

Roorkee (U.P.)
January 1, 1969

Preface to the third edition

In this edition, the book has been revised in general and noteworthy revisions have been made by including new experiments like North Dakota Cone Test for subgrade soil, tests for Joint Filler and Sealer, and tests for Stripping Value of Road Aggregates. Latest recommendations of the Indian Roads Congress have been included. It is hoped that this revised third edition will prove more useful to the students, teachers and the practising engineers. The assistance rendered by Sri S. Venkatesh is thankfully acknowledged.

Authors

Roorkee (U.P.)
April 20, 1977

Preface to the fourth edition

A new experiment, Field Density Test has been added in this edition under Part I of the book. Minor revisions have been made in several other tests, their discussions and applications.

Authors

November 14, 1984

Scope

For the design and construction of highway and airfield, it is imperative to carry out tests on construction materials for their scientific designing and economic utilization. The inherent economy in construction depends upon the maximum use of local materials. The prime objective of the different tests in use is to know and classify the pavement materials into different group depending upon their physical and strength or stability characteristics.

Different types of pavements are constructed on roads for the safe and comfortable movements of various types of vehicles at the desired speeds. All these pavements are laid over a prepared soil surface called subgrade. Therefore it is necessary first to test the properties of the subgrade soil and to evaluate its supporting capacity under the adverse moisture conditions, before taking up the design and construction of the pavements. The basic tests needed on the subgrade soil for this purpose are for the identification and classification of the soil, its density and compaction characteristics. It is also necessary to determine the strength characteristics of the subgrade for the design of pavements. Some of the important tests on subgrade soil are given in Part I of this book.

Pavements are classified into two major groups as flexible and rigid. The pavement sections which do not fall in any one of these types, are being classified as semi-rigid or semi-flexible. Depending upon the technique and the material of construction, the pavements are classified as (i) Water Bound Macadam (ii) Bituminous (iii) Cement Concrete and (iv) Soil Stabilized. In most of the construction techniques, stone aggregates are commonly used. The suitability of road stones are judged by studying their resistance against impact, abrasion, crushing and weathering action. The other properties of aggregates which are important are their shape factor, specific gravity, water absorption, relative affinity with bitumen, etc. The important tests on aggregates for use as a pavement material have been presented in Part II of this book.

In preparing homogeneous mix, the aggregates are bound together in a pavement component layer with binding agents like (i) soil slurry (ii) bituminous materials like emulsion, cutback, bitumen and tar (iii) portland cement. Thus the tests on these binders are also carried out. The use of bituminous pavement for streets, highways and airports has increased quite rapidly during the last few decades. With fast developing petroleum industry in India, more bituminous materials would be available for utilization. Physical tests like penetration, ductility, softening point, flash-fire point etc, are in use in classifying and testing of bituminous materials. Viscosity test is also included in recent years for defining the materials for their scientific use. Part III of this book deals with the tests on bituminous materials.

Bituminous mixes are used in the surface course of pavements; some types of bituminous mixes are also used in the base course of some flexible pavements. These paving mixes are to be designed to withstand the varying conditions of climate and traffic. There are several methods of bituminous mix design. Two of the more common methods are given in Part IV of the book.

Tests on cement and cement concrete mixes are not included here as these are extensively dealt within cement concrete technology through many standard publications.

Most of the tests on soil, aggregates, bituminous materials and mixes are standardised in different countries by different agencies/institutions. The test methods, details and specifications given in this book are based on those given by the Indian Standards Institution (ISI) and the Indian Roads Congress (IRC). However in the case of tests not covered by the ISI or the IRC, the specifications and recommendations of other standard organisations like the British Standards Institution, America n Society for Testing Materials, Asphalt Institute, etc. have been followed.

Contents

Introduction

Soil is very essential highway material because of the under mentioned reasons :

(i) Soil subgrade is part of the pavement structure; further the design and behaviour of pavement especially the flexible pavements depend to a great extent on the subgrade soil.

(ii) Soil is one of the principal materials of construction in soil embankments and in stabilized soil base and sub-base courses.

In view of the wide diversity in soil type, it is desirable to classify the subgrade soil into groups possessing similar physical properties. Many methods have been in use for this purpose. Soil are normally classified on the basis of simple laboratory tests such as grain size analysis and consistency tests.

Soil compaction is important phenomenon in highway construction as compacted subgrade improves the load supporting ability of pavement; in turn resulting in decreased pavement thickness requirement. Compaction of earth embankments would result in decreased settlement. Thus the behaviour of soil sub-grade material could be considerably improved by adequate compaction under controlled conditions. The laboratory compaction test results are useful in specifying the optimum moisture content at which a soil should be compacted and the dry density that should be aimed at the construction site. The in-situ density of prepared subgrade as well as other pavement layers has to be determined by a field density test for checking the compaction requirements and as a field control test for compaction.

There are a number of tests for measuring soil strength; some of them give the strength parameters of the soil, other methods are empirical and give only arbitrary strength values. The types of the strength tests may be classified as shear test, bearing tests and penetration tests. The triaxial test results are useful to find the strength parameters, viz : cohesion and angle of internal friction and modulus deformation of soils. The California Bearing Ratio test is essentially a penetration test which is carried out either in the laboratory or in the field. This test is suitable for the evaluation of strength of soil and aggregates. The method has an important place among highway material testing programme, as it has been extensively correlated with flexible pavement design and performance. North Dakota Cone Test is another penetration test which may also be carried out either in the laboratory or on in-situ soil in the field, but its use is restricted to fine grained soils free from coarse particles. Plate bearing test is carried out either on subgrade to find the modulus of subgrade reaction or on a pavement component layer for pavement evaluation.

There are several soils which are unsuitable as highway material, since they can not be used as such in the base course, sub-base or the subgrade. The strength and durability characteristics of these soils can be improved to the desired extent by adopting a stabilization technique. One of the widely used methods of stabilization is soil-cement which is applicable to a fairly wide range of soil types. The cement stabilized soil can be used in sub-base and base course layers of pavements.

Here in this section test for grain size analysis, consistency limits, soil compaction, triaxial, CBR, NDC and plate bearing tests and mix design of soil-cement are included.

Grain Size Analysis

INTRODUCTION

Most of the methods for soil identification and classification are based on certain physical properties of the soils. The commonly used properties for the classification are the grain size distribution, liquid limit and plasticity index. These properties, have also been used in empirical design methods for flexible pavements; and in deciding the suitability of subgrade soils.

Grain size analysis also known as mechanical analysis of soils is the determination of the percent of individual grain sizes present in the sample. The results of the test are of great value in soil classification. In mechanical stabilization of soil and for designing soil-aggregate mixtures the results of gradation tests are used. Correlations have also been made between the grain size distribution of soil and the general soil behaviour as a subgrade material and the performance such as susceptibility to frost action, pumping of rigid pavements etc. Also permeability characteristics, bearing capacity and some other properties, are approximately estimated based on grain size distribution of the soil.

The soil is generally divided into four parts based on the particle size. The fraction of soil which is larger than 2.00 mm size is called gravel, that between 2.00 and 0.06 mm is sand, between 0.06 and 0.002 mm is silt and that which is smaller than 0.002 mm size is clay. Two types of sieves are available, one type with square perforations on plates to sieve coarse aggregates and gravel, the other type being mesh sieves made of woven wire mesh to sieve finer particles such as fine aggregates and soil fraction consisting of sand silt and clay. However the sieve opening of the smallest mesh sieve commonly available is about 0.075 mm, which is commonly known as 200-mesh sieve or sieve no. 200 as per the British and American specifications. Therefore all soil particles consisting of silt and clay which are smaller than 0.06 mm size will pass through the fine mesh sieve with 0.075 mm opening. Therefore the grain size analysis of the coarser fraction of soil is carried out using sieves and that of finer fraction passing 0.075 mm sieve is carried out using the principle of sedimentation in water.

The mechanical analysis consists of two parts :

(i) the determination of the amount and proportion of coarse material by the use of sieves; and

(ii) the analysis for the fine grained fraction by sedimentation method.

The sieve analysis is a simple test consisting of sieving a measured quantity of material through succesively smaller sieves. The weight retained on each sieve is expressed as a percentage of the total sample. The sedimentation principle has been used for finding the grain size distribution of fine soil fraction; two methods are commonly used viz ; *Pipette method* and the *Hydrometer method*.

In this book only the sieve analysis has been described in detail.

The grain size distribution of soil particles of size greater than 63 micron is determined by sieving the soil on a set of sieves of decreasing sieve opening placed one below the other and separating out the different

size ranges. Two methods of sieve analysis are as follows :

(i) wet sieving applicable to all soils and

(ii) dry sieving applicable only to soils which have negligible proportion of clay and silt.

The soil received from the field is divided into two parts : one, the fraction retained on 2 mm sieve and the other passing 2 mm sieve. The sieve analysis also may be carried out separately for these two fractions. The fraction retained on 2 mm sieve may be subjected to dry sieving using bigger sieves and that passing 2 mm sieve may be subjected to wet sieving; however if this fraction consists of single grained soil with negligible fines passing 0.075 mm size, dry sieving may be carried out.

Apparatus

Various apparatus include set of standard sieves of different sieve sizes, balance, rubber covered pestle and mortar, oven, riffle, sieves shaker.

Procedure

(a) For the fraction retained on 2.0 mm sieve

Sufficient quantity of the dry soil retained on 2.0 mm sieve is weighed out. The quantity of sample taken may be increased when the maximum size of particles is higher. The sample is separated into various fractions by sieving through the set of sieves of sizess 100, 63, 20, 6, 3, 4.75 and 2 mm IS sieves. Additional sieve size may also be introduced if necessary. After initial sieving, the material retained on each sieve is collected, the lumps broken down using mortar and rubber covered pestle and is re-sieved Thus the soil fraction retained on each sieve is carefully collected and weighed.

(b) For the fraction passing 2.0 mm sieve and retained on 0.063 mm sieve

Dry sieving may be done in the case of soils which are cohesionless, single grained and without lumps. The required quantity of soil sample is taken by rifling or quartering method, dried in oven at 105 to 110°C and is subjected to dry sieve analysis using a set of sieves with sieve openings 2.0, 0.6, 0.425, 0.15 and 0.075 mm, pan and lid, additional sieves may be used or any of the sieves removed, depending upon the requirement of the test. The material retained on each sieve and on the pan are separately collected and weighed.

As per the ISI dry sieving is done on the fraction retained on 4.75 mm sieve using 100, 80, 20 and 4.75 mm sieves. Also dry sieving may be done on fraction passing 4.75 mm and retained on 0.075 mm sieve if the soil is not clayey, using 2.0, 0.425 and 0.075 mm sieves.

Wet sieving may be adopted in the case of clayey or cohesive soils or when the soil is not single grained, but has lumps. The soil finer than 2 mm size is oven dried at 105° to 110°C and the required quantity taken by riffling is weighed. This sample is spread in a tray or bucket and covered with water. In case of soils having fractions that are likely to flocculate a dispersing agent like sodium hexametaphosphate (2.0g) or sodium hydroxide (1.0g) and sodium carbonate (1.0 g) per litre of water may be added to the water. The mix is stirred and left for soaking.

The soaked soil specimen is placed over the set of sieves of sizes with the finest sieve and pan at the bottom and washed thoroughly. Washing is continued till the water passing each sieve is substantially clean. The fraction on each sieve is emptied carefully without loss of material in separate trays, oven dried at 105° to 110°C and each fraction weighed separately.

Calculation

The weight of dry soil fraction retained on each sieve is calculated as a percentage of the total dry weight of the sample taken. When a soil contains material coarser as well as finer than 2 mm size, the combined gradation on the basis of total sample should be calculated.

Results

The results are plotted on a semi-logarithmic graph paper with the grain size or sieve size on the X-axis in log. scale and the percentage finer than each size on the Y-axis in ordinary scale (Fig. 1.1). The smooth curve joining the points thus obtained is known as the particle size distribution curve or diagram. Uniformity coefficient of soil is expressed as $\frac{D_{60}}{D_{10}}$ where D_{60} and D_{10} are particle sizes corresponding to 60 and 10 percent finer.

GRAIN SIZE ANALYSIS FOR PORTION PASSING 63 MICRON SIEVE BY SEDIMENTATION

Two methods are in use based on sedimentation principle that the larger grains settle more rapidly than the smaller ones. The Stoke's law is made use of according to which the velocity of settlement of spherical particles is proportional to the square of their diameters. Thus if the depth and the duration of settlement are known, the velocity and hence the diameter of particles at that depth can be estimated. The percentage of particles finer than this diameter should be found using any one of the two methods viz; (i) Pipette method and (ii) Hydrometer method.

Details of the pipette and hydrometer methods are not included in this book as it is beyond the scope of the coverage. For details please refer I.S. 2720 Part IV, Indian Standard Methods of Test for Soils, Grain Size Analysis.

Applications of Grain Size Analysis

(a) Soil classification

In most of the soil classification systems the percentage of material passing 200—mesh and 40—mesh sieve (75 and 420 microns aperture) have been considered from the grain size analysis, though some classification systems use percentage of silt and clay present for classifying the soils. Hence ordinarily the sieve analysis (dry or wet) will be quite sufficient along with tests for consistency limits, for identifying and classifying soils. The two widely accepted soil classification systems for highway engineering purposes are (i) the Highway Research Board (HRB) classification, also known as American Association of State Highway Officials (AASHO) classification or revised Public Roads Administration (PRA) classification and (ii) The Unified classification system, also known as revised Casagrande classification. Both these systems of classification have based their classification on the grain size analysis by sieve analysis, the liquid limit and plasticity index of the soils.

However for knowing the grain size distribution of soils finer than 75 or 63 micron size and to determine the percentage of silt and clay, the sedimentation methods namely the pipette or hydrometer method of test may be adopted.

(b) Grain size distribution

The grain size distribution curve gives the exact idea regarding the gradation of the soils. It is possible to identify whether a soil is well graded uniformly graded, or poorly graded. Uniformity coefficient is also useful to indicate the gradation.

OBSERVATION SHEET

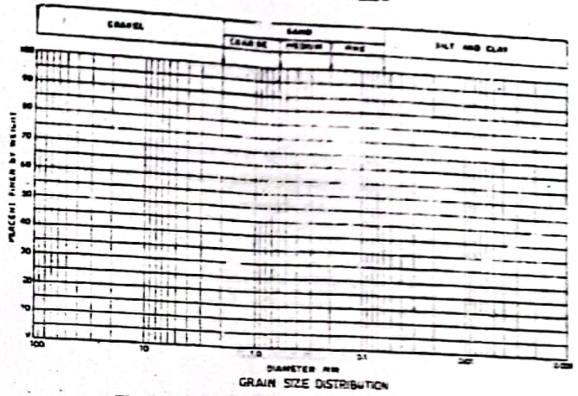

Figure 1.1 Grain Size Distribution Diagram

GRAIN SIZE ANALYSIS—Sieve Analysis

Description of soil :

Type of sieve analysis : dry/wet   Total weight of soil sample, g =

Sieve opening	Weight of sieve or dish, g	Weight of sieve or dish+dry soil, g	Weight of soil retained, g	Cumulative weight retained, g	Cumulative percent retained	Percent Finer
100 mm						
63 ,,						
22 ,,						
6.3 ,,						
4.75,,						
2.0 ,,						
600 micron						
212						
75						
63						
Passing 63 micron						

Report on grain size distribution from the diagram (Figure 1.1)

Uniformity Coefficient = $\frac{D_{60}}{D_{10}}$ =

Report on the gradation of soil :

REMARKS (if any)

(c) Proportioning soils for mix design

In mechanical soil stabilization the main principle is to mix a few selected soils in such a proportion that a desired grain size distribution is obtained for the design-mix. Hence for proportioning the selected soils the grain size distribution of each soil is to be first known.

(d) Group index method of pavement design

See discussion of Experiment 2, Consistency Limits and Indices.

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PROBLEMS

1. Why is soil considered as an important highway material ?
2. Why is wet sieve analysis adopted in some cases and dry sieve analysis in other cases ?
3. How would you interpret the grain size distribution diagram of a given soil ?
4. What is the principle of sedimentation analysis ?
5. Under what circumstances is the sedimentation analysis suggested as part of grain size analysis.
6. What is a deflocculating agent ? What is its function ?
7. What are the types of defloculating agents commonly used in the wet sieve analysis and the sedimentation tests ?
8. Define uniformity coefficient and explain its significance.
9. What are the applications of grain size analysis of soil ?

2

Consistency Limits and Indices

INTRODUCTION

The physical properties of fine grained soils, especially of clay differ much at different water contents. A clay may be almost in liquid state, or it may show plastic behaviour or may be very stiff depending on the moisture content. Plasticity is a property of outstanding importance for clayey soils, which may be explained as the ability to undergo changes in shape without rupture.

Atterberg in 1911 proposed a series of tests, mostly empirical, for the determination of the consistency and plastic properties of fine soils. These are now known as Atterberg limits and indices.

Liquid limit may be defined as the minimum water content at which the soil will flow under the application of a very small shearing force. The liquid limit is usually determined in the laboratory using a mechanical device. An alternate method using a cone penetrometer is also sometimes used for determining the liquid limit of soils.

Plastic limit may be defined in general terms, as the minimum moisture content at which the soil remains in a plastic state. The lower limit is arbitrarily defined and determined in the laboratory by a prescribed test procedure. Plasticity Index (P.I.) is defined as the numerical difference betweeen the liquid and plastic limits. P.I. thus indicates the range of moisture content over which the soil is in a plastic condition.

Shrinkage limit is the maximum moisture content at which further reduction in water content does not cause reduction in volume. It is the minimum water content that can occur in a clayey soil sample which is completly saturated.

Consistency limits and the plasticity index vary for different soil types. Hence these properties are generally used in the identification and classification of soils.

2.1 LIQUID LIMIT TEST

Liquid limit is the moisture content at which 25 blows in standard liquid limit apparatus will just close a groove of standardised dimensions cut in the sample by the grooving tool by a specified amount.

Apparatus

Mechanical liquid limit device consists of a cup and arrangement for raising and dropping through a specified height, as shown in figure 2.1. There are standard grooving tools (a) and (b) as shown. Other apparatus include spatula, evaporating dish, moisture containers, balance of capacity 200 g sensitive to 0.01 g, and oven to maintain 105° to 110° C.

Procedure

About 120 g of dry pulverised soil sample passing 425 micron IS sieve is weighed, and mixed thoroughly with distilled water in the evaporating dish to form a uniform thick paste. The liquid limit device is adjusted to have a free fall of cup through 10 mm. A portion of the paste is placed in the cup above the lowest spot, and squeezed down with the spatula to have a horizontal surface. The specimen is trimmed by firm strokes of spatula in such a way that the maximum depth of soil sample in the cup is 10 mm. The soil in the cup is divided along the diameter through the centre line of the cam followed by firm strokes of the grooving tool so as to get a clean sharp groove as shown in figure 2.2. Grooving tool (b) may be used for all soils, whereas grooving tool (a) may be used only in clayey soils free from sand particles or fibrous materials.

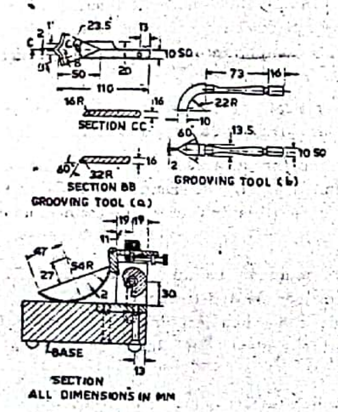

Figure 2.1 Liquid Limit Apparatus and Grooving Tools

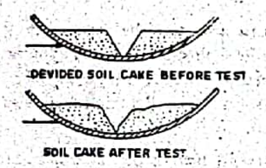

Figure 2.2 Soil Specimen with Groove cut

The crank is rotated at the rate of two revolutions per second (either by hand or electrically depending upon whether it is a hand operated or electrically operated machine) so that the cup is lifted and dropped. This is continued till the two halves of the soil cake come into contact at the bottom of the groove along a distance of about 10 mm, and the number of blows given is recorded. A representative soil is taken, placed in the moisture container, lid placed over it and weighed. The container is dried in oven and the dry weight determined the next day for finding the moisture content of the soil. The operations are repeated for atleast three more trials with slightly increased moisture contents each time, noting the number of blows so that there are atleast four uniformly distributed readings of number of blows between 10 and 40 blows.

Calculations

The flow curve is plotted by taking the number of blows in the log scale on the X—axis, and the water content in arithmetic scale on the Y-axis, see Fig. 2.7 of observation sheet. The flow curve is straight line drawn on this semi-logarithmic plot, as nearly as possible through three or more plotted points. The moisture content corresponding to 25 blows is read from this curve, rounding off to the nearest whole number and is reported as the liquid limit w_l of the soil.

The slope of the straight line flow curve is the flow index. It may be calculated from the following formula :

$$\text{Flow Index, } I_f = \frac{w_1 - w_2}{\log n_2 - \log n_1} = \frac{w_{10} - w_{100}}{\log_{10} 100/10} = w_{10} - w_{100}$$

Hence if the flow curve is extrapolated and moisture content w_{10} and w_{100} corresponding to 10 and 100 blows respectively are found, then the difference in these two water contents would give the flow index of the soil as indicated above.

2.2 PLASTIC LIMIT TEST

Plastic limit is the moisture content at which a soil when rolled into thread of smallest diameter possible, starts crumbling and has a diameter of 3 mm.

Apparatus

Evaporating dish, spatula, ground glass plate, moisture containers, rod of 3 mm diameter, balance sensitive to 0.01 g, drying oven controlled at temperature 105° to 110° C.

Procedure

About 20 g of dry, pulverised soil passing 425-micron IS sieve is weighed out. The soil is mixed thoroughly with distilled water in the evaporating dish till the soil paste is plastic enough to be easily moulded with fingers. A small ball is formed with the fingers and this is rolled between the fingers and the ground glass plate to a thread. The pressure just sufficient to roll into a thread of uniform diameter should be used. The rate of rolling should be between 80 and 90 strokes per minute counting a stroke as one complete motion of hand forward and back to the starting position again. The rolling is done till the diameter of the thread is 3 mm. Then the soil is kneaded together to a ball and rolled again to form thread. This process of alternate rolling and kneading is continued until the thread crumbles under pressure required for rolling and the soil can no longer be rolled into a thread.

If the crumbling starts at diameter less than 3mm, then moisture content is more than plastic limit and if the diameter is greater while crumbling starts, the moisture content is lower. By trial, the thread which starts crumbling at 3 mm diameter under normal rolling should be obtained and this should be immediately transferred to the moisture container, lid placed and weighed to find the wet weight of the thread. The container is kept in the oven for about a day and dry weight found to determine the moisture content of the thread. The above process is repeated to get atleast three consistent values of the plastic limit.

Calculation

The plastic limit (w_p) is expressed as a whole number by obtaining the mean of the moisture contents of the plastic limit.

Plasticity Index (P.I.) is calculated as the difference between liquid limit and plastic limit.

Plasticity Index, I_p = Liquid limit — Plastic limit.

$$= w_L - w_p$$

Toughness Index (I_T) is the ratio of the plasticity index to the flow index and is given by the formula :

$$I_T = \frac{I_p}{I_f}$$

where

I_p is the plasticity index and I_f is the flow index.

Liquidity Index I_l is given by $I_l = \dfrac{w_o - w_p}{I_p}$

where

w_o = natural moisture content of the soil.

Consistency Index, I_c is given by $I_c = \frac{w_p - w_o}{I_p}$

2.3 SHRINKAGE LIMIT TEST

Shrinkage limit, w_s is the maximum water content at which a reduction water content will not cause decrease in volume of the soil. It is also the minimum moisture content to keep a soil saturated without increase in volume.

Apparatus

- Evaporating dish, shrinkage dish of diameter 4.5 cm and height 1.5 cm (both internal) spatula, straight edge, glass cup 5 to 6 cm in internal diameter and 2.5 cm height, two glass plates 7.5 cm × 7.5 cm, one plane and the other having three metal prongs. Also other equipment needed are a 25 ml graduated jar to read 0.2 ml, balance sensitive to 0.01 g, mercury sufficient to fill the glass cup and a dessicator.

Procedure

About 30 g of dry pulverised soil passing 425-micron sieve is weighed out. The soil is placed in the evaporating dish and thoroughly mixed with distilled water to make a paste that may be readily worked into without entrapping air bubbles. The water content to form the paste may be a little more than the liquid limit. The inside of the shrinkage dish is coated with a thin layer of vaseline or heavy grease to prevent adhesion of soil to the dish. The soil paste equal to roughly one third the volume of the dish is placed in the centre of the dish and the paste is allowed to flow to the edges by tapping the dish on a firm surface cushioned with a few layers of blotting paper or similar material. Then another equal quantity of paste is added and tapped so that all the air bubbles entrapped come to the top and the Paste gets compacted. The process is continued till the paste fills the dish completely and starts overflowing. The excess paste is struck off by a straight edge and the outside of the dish is wiped clean.

The dish is immediately weighed and the pat is allowed to dry in air till the colour of the pat becomes lighter. Then the dish is kept in an oven at 105° to 110°C to constant weight, cooled in a dessicator and weighed to find the dry weight. The weight of the empty dish is determined. The volume of the shrinkage dish is found by pouring mercury, until it overflows, removing the excess by pressing the plain glass plate flush with surface of glass cup and measuring the volume of the mercury in the dish by pouring in the graduate.

The glass cup is filled with mercury until it overflows and is pressed with the glass plate having three prongs. This cup full of mercury is placed in a clean evaporating dish, the dry soil pat floated on the mercury and is carefully forced under by the glass plate with prongs. The plate is firmly pressed flush with the surface. Care is taken to ensure that no air is entrapped under the pat. Refer figure 2.3. The volume of mercury displaced is measured in the graduate and this is recorded as the volume of the dry pat.

Calculations

Moisture content of the soil paste taken in the shrinkage dish is calculated.

$$w\% = \left(\frac{W_1 - W_2}{W_2 - W_3}\right) 100 \text{ percent}$$

where W_1, W_2 and W_3, are respectively the weights of dish plus wet soil, dish plus dry soil and dish only.

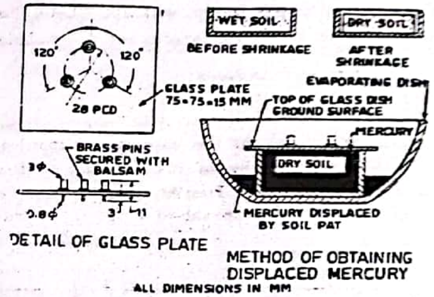

Figure 2.3 Shrinkage Limit Test

Shrinkage limit w_s of remoulded soil is calculated from the relation : $w_s = w - \left(\frac{V - V_o}{W_o}\right) 100$

where, w = moisture content of paste forming wet pat, %

V = Volume of wet pat, cm³ V_o = Volume of the dry pat, cm³

W_o = Weight of oven dry pat = $(W_2 - W_3)$, g

Shrinkage ratio, R is given by, $R = \frac{W_o}{V_o}$

Volumetric shrinkage, V_s is given by, $V_s = (w_1 - w_s)$, $R = (w_1 - w_s)\frac{W_o}{V_o}$

where w_1 = Stipulated moisture content, % ; w_s = Shrinkage limit ; R = Shrinkage ratio

Shrinkage limit can also be calculated just by knowing the oven dry weight and volume of a pat of soil and the specific gravity G of soil particles. This method is especially suitable for finding the shrinkage limit of undisturbed clayey soils.

Shrinkage limit of undisturbed soil is given by, $w_s = \left(\frac{V_o}{W_o} - \frac{1}{G}\right) 100$ percent

Where, V_o = Volume of dry pat of soil.

W_o = Weight of dry pat of soil.

G = Specific gravity of particles.

Discussion

Liquid limit may also be determined by using the cone penetrometer standardised by ISI. The method of test is given in I.S. 2720 part V. The liquid limit of a soil is the moisture content of the paste which would give 25 mm penetration of the standard cone in 30 seconds. This method is fairly rapid when compared with the one carried out by the standard mechanical device.

Determination of plastic limit correctly needs utmost care and experience. Several determinations may be needed before getting three or four reproducible values.

The wet samples collected for moisture content determination during both liquid limit and plastic limit tests should be placed in the moisture container and the lid securely placed immediately before weighing.

Delay in closing the moisture container and in weighing the wet soil sample of soil will cause considerable loss in moisture content of the small quantity of the soil sample, especially during dry weather. One of the main causes for inconsistent results during the liquid limit and plastic limit tests is the error in the determination of moisture content values caused by evaporation before weighing.

Applications of Consistency Limits and Indices

(a) Identification and classification of soils

Liquid limit and plasticity index are two of the important properties for the identification and classification of the fine grained soils. Classification of silty and clayey soils by the HRB and Unified soil classification systems are based on Liquid limit and Plasticity index. Plasticity charts are available with plasticity index plotted against liquid limit. These are shown in figures 2.4 and 2.5.

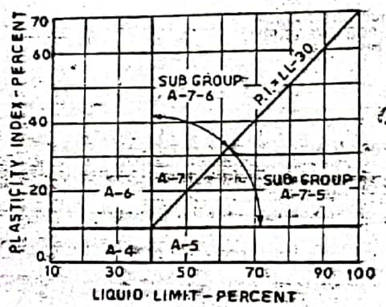

Fig. 2.4 Plasticity Chart for Classifying Fine Grained Soils by HRB Classification System

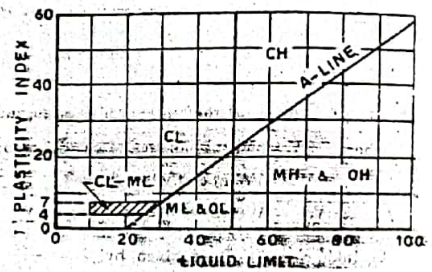

Fig. 2.5 Plasticity Chart for Classifying Fine Grained soils by Unified Soil Classification System

(b) Compressibility.

The liquid limit of a clay indicates its compressibility. Higher the liquid limit, higher is the compressibility. The compression index of *normally consolidated* clay is found to be dependent on the liquid limit. The liquid limit of a soil depends both on the amount and type of clay mineral in the soil.

(c) Plasticity index

This gives an idea about the clay content in a soil. Plasticity index increases with clay content.

(d) Shrinkage limit

This gives an idea about the shrinkage or swelling which is likely to take place after being compacted as a specified moisture content. If a soil is compacted at its OMC which happens to be higher than its shrinkage limit (as in heavy clays) the compacted soil mass will shrink on drying after compaction. If such a clay is compacted at about shrinkage limit (lower than OMC) it is likely to swell on soaking subsequently.

(e) Group index method of pavement design

Higher values of liquid limit and plasticity index (and percent fines passing 200—mesh sieve) are considered as indication of poor subgrade soils. Group index (GI) value is an arbitrary index assigned to soils based on the physical properties, namely, percent passing 200—mesh or 75 micron sieve, liquid limit

and plasticity index and is given by the relation :

$$GI = 0.005\,ac + 0.01\,bd.$$

Here, a = percentage of soil passing 75 micron sieve, greater than 35 and not exceeding 75 percent (value 0—40)

b = percentage of passing 75 micron sieve greater than 15 and not exceeding 55 percent (value 0—40)

c = liquid limit value greater than 40 and not exceeding 60 percent (value 0—20)

d = plasticity index greater than 10 and not exceeding 30 (value 0—20)

The GI value for various soils vary from zero to 20. Higher values of GI indicate inferior subgrade soil which would require greater pavement thickness. Thus GI value is used in the empirical method of pavement design. The GI design chart is given in Figure 2.6.

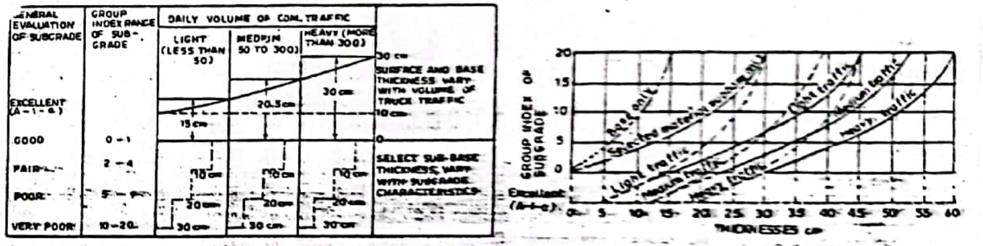

Figure 2.6 Design Chart based on GI value

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PROBLEMS

1. Explain liquid limit, plastic limit and shrinkage limits of soils.
2. Briefly outline the procedure of finding liquid limit of a soil.
3. What is plasticity index ? Explain plasticity property in soils.
4. Define flow index. What is its significance ?
5. What are the various applications of liquid limit ?
6. Liquid limits of soils A and B are 80% and 32% respectively. Which soil is a better highway material ? Why ?
7. The plasticity index of soil P is 6 and that of soil Q is 35. Discuss the suitability of the soils.
8. What are the various uses and applications of consistency limits and indices ?
9. Explain Group index and uses.
10. What are the problems in compacting a heavy clay whose shrinkage limit is much lower than its OMC.

OBSERVATION SHEET

LIQUID LIMIT, PLASTIC LIMIT AND SHRINKAGE LIMIT TESTS

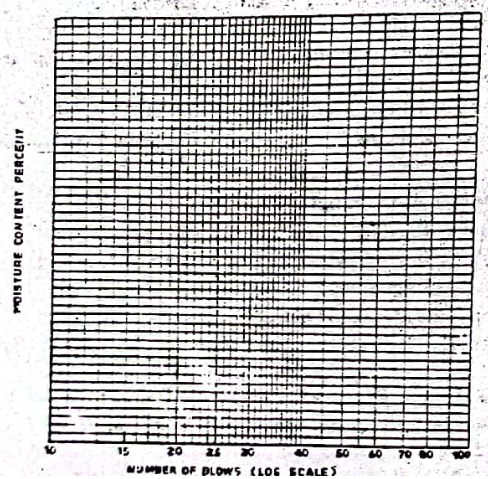
Soil Type :

Liquid Limit :

Natural water content, % =

Determination Number	1	2	3	4	5	6
Number of blows						
Moisture container number ...						
Weight of container, g						
Weight of container+wet soil, g ...						
Weight of container+oven dry, soil, g ...						
Weight of water, g						
Weight of dry soil, g						
Moisture content, %						

Liquid Limit = Moisture content at 25 blows [From the graph]



OBSERVATION SHEET (Contd.)

Plastic Limit :

Determination Number	1	2	3	4	5
Moisture container number					
Weight of container, g					
Weight of container+wet soil, g ...					
Weight of container+dry soil, g ...					Mean Value
Weight of water, g ...					
Weight of dry soil, g					
Moisture content, %					

Plastic Limit =

Results :—Liquid Limit, w_l =

Plastic Limit, w_p =

Toughness Index, I_T =

Liquidity Index, I_L =

Flow Index, I_f =

Plasticity Index I_p =

Consistency Index, I_c =

Report :—Type of soil :

Shrinkage Limit :

Determination Number	1	2	3	4
Weight of shrinkage dish, g				
Weight of dish+wet soil pat, g ...				
Weight of dish+dry soil pat, g ...				
Weight of wet pat, (W)g ...				
Weight of dry soil pat (W_s), g ...				
Moisture content of soil pat w% ...				Mean Value
Volume of wet soil pat (V), cm^3 = Volume of dish ...				
Volume of dry soil pat (V_0), cm^3				
Shrinkage Limit (w_s), %				
Shrinkage ratio (R)				
Volumetric shrinkage (V_s)				
Linear shrinkage (L_s)				

Remarks :

3

Compaction Test

INTRODUCTION

Compaction of soil is a mechanical process by which the soil particles are constrained to be packed more closely together by reducing the air voids. Soil compaction causes decrease in air voids and consequently an increase in dry density. This may result in increase in shearing strength. The possibility of future settlement or compressibility decreases and also the tendency for subsequent changes in moisture content decreases. Degree of compaction is usually measured quantitatively by dry density.

Increase in dry density of soil due to compaction mainly depends on two factors—(i) the compacting moisture content and (ii) the amount of compaction. For practically all soils it is found that with increase in the compaction moisture content, the dry density first increases and then decreases if compacted by any method. This indicates that under a given compactive effort every soil has an optimum moisture content (OMC) at which the soil attains maximum dry density. This fact was first recorded by R. R. Proctor in 1933.

For a soil at a given moisture content, increasing amounts of compaction result in closer packing of soil particles and an increase in dry density, until the volume of air voids is so decreased that further compaction produces no substantial change in the volume. It has been found that in all soils, with all methods of compaction, increase in compacting energy applied per unit weight of soil result in an increase in maximum dry density and a decrease in OMC. In the field, compaction may be carried out by (i) applying pressure on soil layers by means of rollers (ii) ramming (iii) vibration (iv) watering, depending upon the soil type and nature of the project.

In the laboratory various types of compacting equipment and test methods have been developed for determining the moisture—density relationships of soils. These tests may be classified as :

(i) Static compaction test, an example of a test method of this type is California static load compaction developed at California Division of Highway.

(ii) The dynamic compaction tests, which are commonly adopted tests in the laboratory, the test methods which are often followed are (a) Proctor Test, later standardised by different agenices and known as BS compaction IS light compaction and Standard AASHO tests (b) modified AASHO tests also standardised by ISI and known as IS heavy compaction (c) the indirect application of impact through a piston as in the Dietert tests or Iowa Bearing value apparatus.

(iii) Kneading compactor in which the compaction is achieved by applying a gradually increasing stress through rounded end of a piston and releasing it gradually after retaining it for a small interval of time. This is considered to simulate the type of compacting process as rolling in the field.

It has been found that the stress-strain behaviour of a compacted soil by different methods of compaction is likely to vary even if the compacting moisture content and dry density values are the same.

In this chapter, laboratory test using dynamic compaction has been explained. The test is divided into two parts (i) light compaction which is similar to the BS or Standard AASHO compaction test and (ii) heavy

compaction which is similar to the modified AASHO compaction test. These two have been standardised by the ISI (I.S. 2720 parts VII and VIII).

Apparatus

(a) Cylindrical mould of capacity 1000 cc. with an internal diameter of 10 cm and height 12.73 cm or a mould of capacity 2250 cc, with an internal diameter of 15 cm and height of 12.73 cm. The mould is fitted with a detachable base plate and removable collar or extension of about 6 cm high.

(b) For light compaction, a metal rammer having 5 cm diameter circular face, and weight 2.6 kg is used which has a free drop of 31 cm.

For heavy compaction, the rammer has 5 cm diameter circular face, but having weight 4.89 kg and free drop of 45 cm.

(c) Steel straight edge having bevelled edge for trimming the top of the specimen.

(d) Other accessories include moisture containers, balances of capacity 10 kg and 200 g oven, sieves and mixing tools.

Procedure

In case the soil sample has particles bigger than 4.75 mm sieve, about 20 kg of the representative soil is air dried, mixed pulverised and sieved through 20 mm and 4.75 mm sieves. The fraction retained on 20 mm sieve is not used in this test. The percent passing 20 mm sieve and retained on 4.75 mm sieve is noted and if this is less than 20 percent this sample is used as such. If it is more than 20 percent this proportion is maintained every time in repeated tests.

In case the sample passes 4.75 mm sieve, then the dry pulverised sample is sieved through 4.75 mm sieve and the portion passing this sieve is only used for this test. About 16 kg of dry soil in total may be necessary for the compaction test if the 1000 cc mould is used and about 35 kg if the 2250 cc mould is used. For compacting the soil in the mould every time the required quantity will depend on the soil type, size of the mould, moisture content and amount of compaction. As a rough guidance, for each test 2.5 kg of soil may be taken for light compaction and 2.8 kg for heavy compaction, and then the required water is added in the case of 1000 cc mould. In the case of 2250 cc mould the weights may be 6.3 and 7 kg. respectively. The estimated weight of water to be added to the soil every time may be measured in a jar graduated in cc.

Enough water is added to the specimen to bring the moisture content to about 7% less than the estimated O.M.C. for sandy soils and 10% less for clayey soils. The processed soil is stored in an air tight container for about 18 to 20 hours to enable moisture to spread uniformly in the soil mass.

The mould with base plate fitted in is weighed. The process soil-water mixture is mixed thoroughly and divided into eight equal parts.

(i) For light compaction the wet soils is compacted into the mould in three equal layers, each layer being given 25 blows of the 2.6 kg hammer, if 10 cm diameter mould is used. When the 15 cm diameter mould is used, 56 blows are given to each of the three layers by the 2.6 kg hammer.

(ii) For heavy compaction, the wet soil mix is compacted in the mould in five equal layers each layer being given 25 blows of 4.89 kg hammer when the 10 cm diameter mould is used. When the 15 cm diameter mould is used, 56 blows are given to each of the five layers by 4.89 kg rammer.

The blows should be uniformly distributed over the surface of each layer. Each layer of the compacted soil is scored with a spatula before placing the soil for the succeeding layer. The amount of soil used should be just sufficient to fill the mould leaving about 5 mm to be struck off on the top after compacting the final layer.

The collar is removed and the compacted soil is levelled off to the top of the mould by means of the straight edge. The mould and the soil is then weighed. The soil is then ejected out of the mould and cut in the middle and a representative sample is taken in air tight container from the cut surface. The moisture content of this representative specimen is determined by finding the wet weight, keeping in the oven at 105°C-110°C and finding the dry weight the next day.

This procedure is repeated five to six times using fresh part of the soil specimen and after adding a higher water content than the preceding specimen every time so that the last compaction is carried out at moisture 7 to 10 percent higher than the estimated optimum moisture content.

Calculation

Let the weight of mould with moist compacted soil be = W g

Weight of empty mould = W_m g Volume of the mould = V_m cc

Wet density, $\gamma_m = \dfrac{W-W_m}{V_m}$ g/cc Let the moisture content be = w%

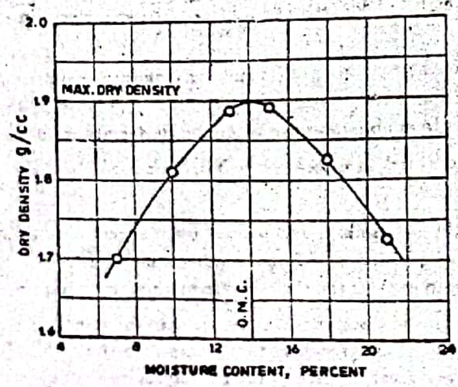
Then dry density, $\gamma_d = \dfrac{\gamma_m}{\left(1+\dfrac{w}{100}\right)} = \dfrac{(W-W_m)}{V_m\left(1+\dfrac{w}{100}\right)}$ g/cc $\gamma_d = (1-n)G\gamma_w$

Porosity $n = 100\dfrac{V_v}{V}\% = \left(1-\dfrac{\gamma_d}{G\gamma_w}\right)100\%$ where G is specific gravity of the soil particles

Void ratio $e = \left(\dfrac{G\,\gamma_w}{\gamma_d} - 1\right)$

Results

Points are plotted with moisture content on the X-axis and dry density on the Y-axis and a smooth curve is drawn connecting the points. From this curve, the maximum dry density is noted and the corresponding value of moisture content taken as optimum moisture content of the soil. Refer figure 3.1.



Discussion

It is necessary to control the total volume of soil compacted every time, as the results will be inaccurate if the amount of soil struck off after removing the extension is too great or is very different every time.

Hence the quantity of soil in each layer should be adjusted in such a way that after removing th extension about 5 mm soil remains to be struck off every time so that the compactive effort or energy spen per unit volume is kept constant. The compacted thickness of each layer should also be equal.

Applications of Compaction Test

From the compaction test, the maximum dry density and optimum moisture content of the soil is foun for the selected type and amount of compaction. These results have various uses.

(a) The OMC of the soil indicates the particular moisture content at which the soil should i compacted to achieve maximum dry density. If the compacting effort applied is less, the OMC increas and the value can again be found experimentally or estimated.

(b) In field compaction, the compacting moisture content is first controlled at OMC and the adequa of rolling or compaction is controlled by checking the dry density achieved and comparing with t maximum dry density achieved in the laboratory. Thus compaction test results (OMC and maximum d density) are used in the field control test in compaction projects.

Compaction, in general is considered most useful in the preparation of subgrade and other pavem layers and in construction of embankments in order to increase the stability and to decrease settlem There is also a soil classification method based on the maximum dry density in the standard (Proc compaction test, the lower values indicating weaker soils. According to the method suggested K. B. Woods, soils with maximum values of dry density (in g/cm³) greater than 2.1 are excellent, 1.9 to are good, 1.75 to 1.90 are fair, 1.60 to 1.75 are poor and those less than 1.60 are very poor.

REFERENCES

1. Soil Mechanics for Road Engineers, *D.S.I.R.*, H.M.S.O., London.
2. Procedure for Testing soils, *American Society for Testing Materials*, 1950
3. Soil Testing for Engineers, *T.W. Lambe*, John Wiley and Sons, New York.
4. Methods of Test for soil. IS : 2720, *Indian Standards Institution*.

PROBLEMS

1. Explain how soil compaction is achieved.
2. What are the effects of compaction of soils ?
3. Explain optimum moisture content.
4. What are the various types of laboratory compaction tests ?
5. How can standard compaction test be useful in soil classification ?
6. What are the applications of compaction test results in highway construction ?

soil used should be just sufficient to fill the mould leaving about 5 mm to be struck off on the top after compacting the final layer.

The collar is removed and the compacted soil is levelled off to the top of the mould by means of the straight edge. The mould and the soil is then weighed. The soil is then ejected out of the mould and cut in the middle and a representative sample is taken in air tight container from the cut surface. The moisture content of this representative specimen is determined by finding the wet weight, keeping in the oven at 105°C-110°C and finding the dry weight the next day.

This procedure is repeated five to six times using fresh part of the soil specimen and after adding a higher water content than the preceding specimen every time so that the last compaction is carried out at moisture 7 to 10 percent higher than the estimated optimum moisture content.

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Let the weight of mould with moist compacted soil be = W g

Weight of empty mould = W_m g

Volume of the mould = V_m cc

Wet density, $\gamma_m = \dfrac{W - W_m}{V_m}$ g/cc

Let the moisture content be = w%

Then dry density, $\gamma_d = \dfrac{\gamma_m}{\left(1+\dfrac{w}{100}\right)} = \dfrac{(W - W_m)}{V_m\left(1+\dfrac{w}{100}\right)}$ g/cc

Porosity $n = 100\dfrac{V_v}{V}\% = \left(1 - \dfrac{\gamma_d}{G\gamma_w}\right)100\%$

where G is specific gravity of the soil particles

Void ratio $e = \left(\dfrac{G.\gamma_w}{\gamma_d} - 1\right)$

Results

Points are plotted with moisture content on the X-axis and dry density on the Y-axis and a smooth curve is drawn connecting the points. From this curve, the maximum dry density is noted and the corresponding value of moisture content taken as optimum moisture content of the soil. Refer figure 3.1.

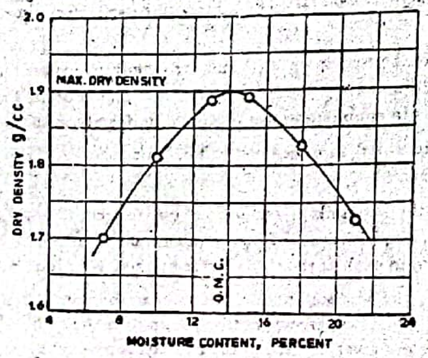

Figure 3.1 Compaction Curve

Discussion

It is necessary to control the total volume of soil compacted every time, as the results will be inaccurate if the amount of soil struck off after removing the extension is too great or is very different every time.

Hence the quantity of soil in each layer should be adjusted in such a way that after removing the extension about 5 mm soil remains to be struck off every time so that the compactive effort or energy spent per unit volume is kept constant. The compacted thickness of each layer should also be equal.

Applications of Compaction Test

From the compaction test, the maximum dry density and optimum moisture content of the soil is found for the selected type and amount of compaction. These results have various uses.

(a) The OMC of the soil indicates the particular moisture content at which the soil should be compacted to achieve maximum dry density. If the compacting effort applied is less, the OMC increases and the value can again be found experimentally or estimated.

(b) In field compaction, the compacting moisture content is first controlled at OMC and the adequacy of rolling or compaction is controlled by checking the dry density achieved and comparing with the maximum dry density achieved in the laboratory. Thus compaction test results (OMC and maximum dry) density) are used in the field control test in compaction projects.

Compaction, in general is considered most useful in the preparation of subgrade and other pavement layers and in construction of embankments in order to increase the stability and to decrease settlement. There is also a soil classification method based on the maximum dry density in the standard (Proctor) compaction test, the lower values indicating weaker soils. According to the method suggested by K. B. Woods, soils with maximum values of dry density (in g/cm³) greater than 2.1 are excellent, 1.9 to 2.1 are good, 1.75 to 1.90 are fair, 1.60 to 1.75 are poor and those less than 1.60 are very poor.

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PROBLEMS

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5. How can standard compaction test be useful in soil classification ?
6. What are the applications of compaction test results in highway construction ?

OBSERVATION SHEET

COMPACTION TEST

Volume of the mould, Vm cc = ; Amount of compaction = Light/Heavy
Weight of the rammer = ; Type of soil =
Specific gravity of soil G = ;

percentage material retained on 20 mm sieve =
passing 20 mm and retained on 4.75 mm sieve =
passing 4.75 mm sieve =

Determination number	1	2	3	4	5	6	7
Weight of the mould $= W_m$, g							
Weight of mould+compacted soil $= W$ g							
Moisture container number							
Weight of Moisture container $= W_1$ g							
Weight of container + wet soil $= W_2$ g							
Weight of container + dry soil $= W_3$ g							
Wet density $\gamma_m = \frac{(W-W_m)}{V_m}$ g/cc							
Moisture content = $w\% = \frac{(W_2-W_3)\,100}{(W_3-W_1)}$							
Dry density, $\gamma_d = \frac{\gamma_m}{1+\frac{w}{100}}$ g/cc							
Void ratio, $e = \left(\frac{G\,\gamma_w}{\gamma_d}\right) - 1$							
Porosity, $n = \left(1-\frac{\gamma_d}{G\,\gamma_w}\right)\%$							

From graph :

Maximum dry density = g/cc

Optimum moisture content = %

Remarks

4

Field Density Test by Sand Replacement Method

INTRODUCTION

The dry density of compacted soil or pavement material is a common measure of the amount of the compaction achieved during the construction. Knowing the field density and field moisture content, the dry density is calculated. Therefore field density or in-situ density test is of importance as a field control test for the compaction of soil or any other pavement layer. The determination of field density of natural bed of soil has also various other applications in civil engineering.

There are several methods for the determination of field density of soils such as core cutter method, sand replacement method rubber, baloon method, heavy oil method, etc. One of the common methods of determining field density of fine grained soils is core cutter method; but this method has a major limitation in the case of soils containing coarse grained particles such as gravel, stones and aggregates. Under such circumstances, field density test by sand replacement method is advantageous as the presence of coarse grained particles will adversely affect the test results and also the method is quite simple.

The basic principle of sand replacement method is to measure the in-situ volume of whole from which the material was excavated from the weight of sand with known density filling in the hole. The in-situ density of material is given by the weight of the excavated material divided by the in-situ volume.

Apparatus

(a) Sand pouring Cylinder : This consists of a metal cylinder of capacity three litres, 115 mm in diameter and 380 mm in length with an inverted funnel or cone at one end and a shutter to open and close the entry of sand and a cap on the other end. This cylinder is considered suitable to determine the field density of fine grained soils (not less than 90 percent passing 2.0 mm sieve) and medium grained soils (not less than 90 percent passing 20 mm sieve). However a larger size cylinder is suggested for the determination of field density of coarse grained soils.

(b) Metal tray with hole : A metal tray, 300 mm square and 40 mm deep with a 100 mm diameter hole at the center to enable excavation of a hole of proper size and shape.

(c) Tools for levelling and excavating : Hand tools such as scraper with handle for levelling the surface; a dibber or an elongated trowel for digging and excavating the material.

(d) Containers : Metal containers of any convenient size (about 150 mm diameter and 200 mm depth) with removable lid for collecting the excavated material.

(e) Calibrating container : A metal cylinder of internal diameter 100 mm and depth 150 mm with a flange of width 50 mm, to determine the weight and volume of sand for the purpose of calibration. The sand is to be poured into the calibrating container through the sand pouring cylinder.

(f) Plane surface : A glass or perspex plate, about 450 mm square to serve as a plane surface for finding the weight of sand occupying the cone.

(g) Balance : A suitable balance of capacity 10 or 15 kg and accuracy 1.0 g and necessary set of weights.

(h) Sand : Dry and clean test sand of uniform gradation, passing 1.0 mm and retained 0.6 mm sieve.

...iner and dibber are shown in Fig. 4.1.

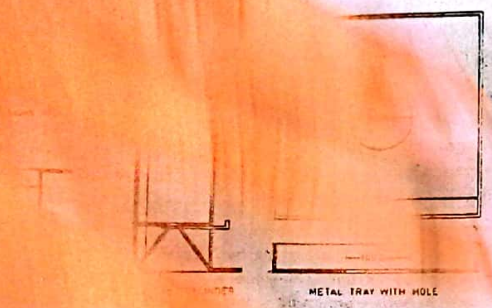

Fig. ... nder Apparatus

...he test may be conducted in two stages : (i) ... ratus and measurement of in-situ density.

Calibration of apparatus

The determination of volume of the excavated hole is based on the weight of sand filling the hole and the cone and the density of the sand. Calibration of apparatus includes (a) determination of density of test sand used in the experiment under identical height and pouring conditions of the sand into the test hole and (b) determination of the weight of the sand occupying the cone of the sand pouring cylinder.

Clean and dry test sand passing 1.0 mm sieve and retained on 0.6 mm sieve is collected in sufficient quantity required for atleast three to four set of tests (say, atleast 14 to 15 kg in weight). The top cap of the sand pouring cylinder is removed, the shutter is closed, the cylinder is filled with dry test sand upto about 10 mm from the top and the cap is replaced. The weight of the cylinder with the sand is determined accurate to one gram and is recorded = W_1. In all the subsequent tests for calibration as well as for the field density tests, every time the sand is filled into the cylinder such that the initial weight of the cylinder with sand is exactly W_1. The sand pouring cylinder is placed over the calibration cylinder or one of the test holes already excavated, the shutter is opened and the sand equal to the volume of the calibration cylinder or the excavated test hole is allowed to flow out and the shutter is closed.

The sand pouring cylinder is now placed on a clean plane surface (glass or perspex plate), the shutter is kept open till the sand fills up the cone fully and there is no visible movement of sand as seen from the top of the cylinder by removing the cap. The shutter is closed, the cylinder is removed and the sand which occupied the cone is carefully collected from the plate and weighed = W_2.

The sand pouring cylinder is refilled with sand such that the initial weight is again W_1. Now the cylinder is placed centrally on the top of the calibrating container and the shutter is opened. When the sand fills up the calibrating container and the cone completely and there is no movement of sand, the shutter is closed and the sand pouring cylinder and the remaining sand is weighed = W_3.

The above steps are repeated three times and the mean values of W_2 and W_3 are determined such that the mean value of the weight of sand required to fill the calibrating container upto the level top can be determined.

The volume of the calibrating container, V is determined either by measuring the internal dimensions (diameter and height) or by filling with water and weighing. From the weight of sand W_s and its volume V in the calibrating container, the density of sand, is determined.

Measurement of field density

The site where the field density test is to be conducted is cleaned and levelled using a scraper for an area of about 450 mm square. The metal tray with central hole is placed on the prepared surface. Using this central hole as pattern, the soil/material is excavated using a dibber or a trowel upto a depth of 150 mm and the loose material removed is carefully collected in the metal container and is weighed = W. The sand pouring cylinder is refilled with sand such that its weight is again W_1. The metal tray with central hole is removed and the sand pouring cylinder is placed centrally over the excavated hole. The shutter is opened till the sand fills the excavated hole and the cone completely and there is no further movement of sand in the cylinder. The shutter is closed and the cylinder is weighed again = W_4, so that the weight of sand filling the excavated hole alone = W_b can be found.

The moisture content of the excavated soil, w % is determined by taking a sample of soil from it in a moisture content dish, weighing, drying in oven at 110°C and re-weighing. Alternatively, the moisture content (w %) is determined by placing the entire excavated soil collected from the hole (of weight W) in the oven and finding its dry weight = W_d.

The above steps for the determination of the weights of excavated soil, the weight of the sand filling the hole and the weights of samples for the moisture content determination are repeated atleast three times and the average values taken for the determination of field density (wet and dry) values.

Calculation and Results

W_1 = weight of sand pouring cylinder and sand filled upto 10 mm from top edge; g; whenever the sand is refilled in the cylinder, the total weight should be exactly W each time, throughout the experiment.

W_2 = weight of sand in the cone, mean value, g

W_3 = weight of cylinder and sand after pouring into the calibrating container and cone, mean value, g.

W_4 = weight of cylinder and sand after pouring into the excavated hole and cone, mean value, g

V_s = volume of the excavating container, cm^3

W = weight of the soil from the excavated hole, mean value, g

W_d = oven dry weight of the soil excavated from the hole, mean value, g

w = moisture content of the soil, mean value, %

The weight of sand filling the calibrating container only = $W_s = (W_1 - W_3 - W_2)$, g

(i) Bulk density of sand, $\gamma_s = \frac{W_s}{V_s}$, g/cm³

Weight of sand filling the excavated hole alone = $W_1 - W_4 - W_2 = W_b$, g

Volume of sand filling the excavated hole alone = $\frac{W_b}{\gamma_s} = V$, cm³

(ii) In-situ bulk density of wet excavated soil, $\gamma = \frac{W}{V}$, g/cm³

(iii) Moisture content of soil, $w\ \% = \frac{100\,(W - W_d)}{W_d}$, %

(iv) In-situ dry bulk density of the excavated soil, $\gamma_d = \frac{\gamma W_d}{W}$, g/cm³ $= \frac{100\,\gamma}{(100 + W)}$, g/cm³

The sand pouring cyclinder, metal tray, calibrating container and dibber are shown in Fig. 4.1.

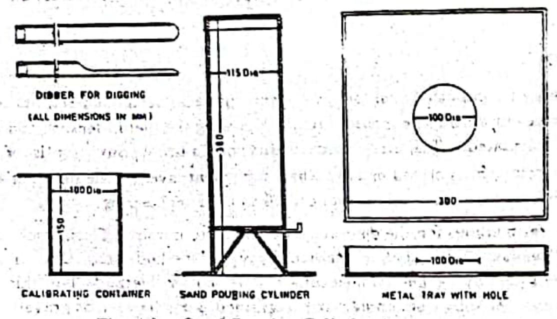

Fig. 4.1 Sand Pouring Cylinder Apparatus

Procedure

The test may be conducted in two stages : (i) calibration of apparatus and measurement of in-situ density.

Calibration of apparatus

The determination of volume of the excavated hole is based on the weight of sand filling the hole and the cone and the density of the sand. Calibration of apparatus includes (a) determination of density of test sand used in the experiment under identical height and pouring conditions of the sand into the test hole and (b) determination of the weight of the sand occupying the cone of the sand pouring cylinder.

Clean and dry test sand passing 1.0 mm sieve and retained on 0.6 mm sieve is collected in sufficient quantity required for atleast three to four set of tests (say, atleast 14 to 15 kg in weight). The top cap of the sand pouring cylinder is removed, the shutter is closed, the cylinder is filled with dry test sand upto about 10 mm from the top and the cap is replaced. The weight of the cylinder with the sand is determined accurate to one gram and is recorded = W_1. In all the subsequent tests for calibration as well as for the field density tests, every time the sand is filled into the cylinder such that the initial weight of the cylinder with sand is exactly W_1. The sand pouring cylinder is placed over the calibration cylinder or one of the test holes already excavated, the shutter is opened and the sand equal to the volume of the calibration cylinder or the excavated test hole is allowed to flow out and the shutter is closed.

The sand pouring cylinder is now placed on a clean plane surface (glass or perspex plate), the shutter is kept open till the sand fills up the cone fully and there is no visible movement of sand as seen from the top of the cylinder by removing the cap. The shutter is closed, the cylinder is removed and the sand which occupied the cone is carefully collected from the plate and weighed = W_2.

The sand pouring cylinder is refilled with sand such that the initial weight is again W_1. Now the cylinder is placed centrally on the top of the calibrating container and the shutter is opened. When the sand fills up the calibrating container and the cone completely and there is no movement of sand, the shutter is closed and the sand pouring cylinder and the remaining sand is weighed = W_3.

The above steps are repeated three times and the mean values of W_2 and W_3 are determined such that the mean value of the weight of sand required to fill the calibrating container upto the level top can be determined.

The volume of the calibrating container, V is determined either by measuring the internal dimensions (diameter and height) or by filling with water and weighing. From the weight of sand W_s and its volume V in the calibrating container, the density of sand, is determined.

Measurement of field density

The site where the field density test is to be conducted is cleaned and levelled using a scraper for an area of about 450 mm square. The metal tray with central hole is placed on the prepared surface. Using this central hole as pattern, the soil/material is excavated using a dibber or a trowel upto a depth of 150 mm and the loose material removed is carefully collected in the metal container and is weighed = W. The sand pouring cylinder is refilled with sand such that its weight is again W_1. The metal tray with central hole is removed and the sand pouring cylinder is placed centrally over the excavated hole. The shutter is opened till the sand fills the excavated hole and the cone completely and there is no further movement of sand in the cylinder. The shutter is closed and the cylinder is weighed again = W_4, so that the weight of sand filling the excavated hole alone = W_b can be found.

The moisture content of the excavated soil, w % is determined by taking a sample of soil from it in a moisture content dish, weighing, drying in oven at 110°C and re-weighing. Alternatively, the moisture content (w %) is determined by placing the entire excavated soil collected from the hole (of weight W) in the oven and finding its dry weight = W_d.

The above steps for the determination of the weights of excavated soil, the weight of the sand filling the hole and the weights of samples for the moisture content determination are repeated atleast three times and the average values taken for the determination of field density (wet and dry) values.

Calculation and Results

W_1 = weight of sand pouring cylinder and sand filled upto 10 mm from top edge; g; whenever the sand is refilled in the cylinder, the total weight should be exactly W each time, throughout the experiment.

W_2 = weight of sand in the cone, mean value, g

W_3 = weight of cylinder and sand after pouring into the calibrating container and cone, mean value, g.

W_4 = weight of cylinder and sand after pouring into the excavated hole and cone, mean value, g

V_s = volume of the excavating container, cm³

W = weight of the soil from the excavated hole, mean value, g

W_d = oven dry weight of the soil excavated from the hole, mean value, g

w = moisture content of the soil, mean value, %

The weight of sand filling the calibrating container only = $W_s = (W_1 - W_3 - W_2)$, g

(i) Bulk density of sand. $\gamma_s = \frac{W_s}{V_s}$, g/cm³

Weight of sand filling the excavated hole alone = $W_1 - W_4 - W_2 = W_b$, g

Volume of sand filling the excavated hole alone = $\frac{W_b}{\gamma_s} = V$, cm³

(ii) In-situ bulk density of wet excavated soil, $\gamma = \frac{W}{V}$, g/cm³

(iii) Moisture content of soil, $w\ \% = \frac{100\,(W - W_d)}{W_d}$, %

(iv) In-situ dry bulk density of the excavated soil, $\gamma_d = \frac{\gamma W_d}{W}$, g/cm³ $= \frac{100\,\gamma}{(100 + W)}$, g/cm³

The results are reported as the average value of atleast three sets of tests in the following :

(i) In-place wet density of soil in g/cm^3, correct to second decimal place or in kg/m^3, correct to nearest whole number.

(ii) In-place dry density of soil in g/cm^3 or in kg/m^3 (as above).

(iii) Moisture content of the soil in percent, correct to first decimal place.

Discussion

Care should be taken to see that the test sand used is clean, dry and is uniformly graded (passing 1.0 mm sieve and retained on 0.6 mm sieve). Moist and unclean sand does not flow freely through the opening of the cone and does not fill up the hole and the cone completely. The density of the test sand will get affected by the height of free fall of the sand, vibrations, etc. Therefore the initial weight of the sand pouring cylinder and the sand is maintained the same = W_1 throughout the repeated tests of the experiment, both during the calibration for the determination of the density of the sand and during determination of the volume of the excavated hole. The depth of the excavated hole should be equal to the depth of the calibrating container (150 mm). There should be no disturbance or vibration caused by tapping the cylinder, during the test. The sand poured into the excavated hole should not be normally re-used without cleaning, sieving and drying again, as it is likely to get mixed with the soil in the hole.

When the in-situ density of soil/materials with grain size larger than 20 mm is to be determined, larger size sand puring cylinder of diameter 215 mm and height 610 mm (capacity 16.5 litres) is used. The calibration container should be 200 mm in diameter and 250 mm in height; the hole in the metal tray should be 200 mm in diameter and the depth of excavated hole should be 250 mm.

Applications of the field density test

Determination of field density or in-place density, moisture content and dry density of a compacted layer of soil or pavement layer are often used to check the amount of compaction attained in the field and for quality control tests in field compaction.

The determination of in-situ density of soil is useful in calculating the over burden pressure of a soil deposit, calculation of bearing capacity, analysis of stability slopes and foundations and in several other problems. The dry density of a compacted soil mass is also used for assessing the soil type and its stability.

The sand replacement method of field density test can be applied for any type of soil and pavement materials, whereas the core cutter method can not be used if coarse particles are present in the compacted layer.

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2. Soil Mechanics for Road Engineers *DSIR*, HMSO, London.

PROBLEMS

1. What is the principle of sand replacement method of determining in-situ density of a compacted layer ?
2. What are the objects of calibration test in sand replacement method ?
3. How is the wet density of soil calculated from the test data ?
4. How is the dry density of soil determined knowing the wet density and moisture content ?
5. What are the sources of error in sand replacement method ? What are the precautions to be taken ?
6. What are the advantages of sand replacement method as compared to core cutter method ?

OBSERVATION SHEET

DETERMINATION ON FIELD DENSITY BY SAND REPLACEMENT METHOD

Size of sand pouring cylinder : Standard size (three kg)/large size

Type of material for density determination :

Observations and calculations	Test number 1	2	3	Mean
(a) Calibration test :				
1. Weight of sand pouring cylinder and full sand = W_1, g (to be maintained same every time)				
2. Weight of sand in cone, W_2, g				
3. Volume of calibrating container = V_a, cm^3				
4. Wt. of cylinder and sand after filling calibrating container = W_3, g				
5. Wt. of sand filling calibrating containor and cone = W_a = $(W_1 - W_3 - W_2)$, g				
6. Bulk density of sand, $\gamma_s = \frac{W_a}{V_a}$, g/cm^3				
(b) Field density test :				
7. Wt. of sand pouring cylinder with full sand = W_1, g				
8. Wt. of moist soil/material from excavated hole = W, g				
9. Wt. of cylinder and sand after filling the excavated hole = W_4, g				
10. Wt. of sand in the excavated hole = W_b, g = $(W_1 - W_4 - W_2)$, g				
11. Volume of sand filling the excavated hole, $V = \frac{W_b}{\gamma_s}$, cm^3				
12. In-situ bulk density of wet excavated soil, $\gamma = \frac{W}{V}$, g/cm^3				
13. (i) Weights of soil samples for moisture content determinations : Wet Wt. = Dry Wt. =				
Or (ii) Weight of soil/material excavated from the hole, after drying = W_d, g				
14. Moisture content of excavated soil, w, % =				
15. In-situ bulk density of dry soil, γ_d g/cm^3				
(i) $= \frac{100\,\gamma}{(100 + w)}$, g/cm^3				
or (ii) $= \frac{\gamma W_d}{W}$, g/cm^3				

Remarks :

5

Triaxial Test

INTRODUCTION

Shear tests are generally carried out on small soil samples in the laboratory to evaluate the strength properties of the element in the soil mass. The strength parameters, namely the cohesion and angle of shearing resistance are usually found from these tests. The two methods of shear tests commonly used are the *direct shear* test and the *triaxial* test. In the direct shear test the failure is caused along a pre-determined plane of the soil and both the shearing resistance and normal stress are measured directly. In triaxial test a cylindrical specimen is stressed in lateral and vertical directions and the shear strength of the soil is evaluated; the plane of shear failure is not pre-determined. The triaxial test is considered as much superior strength test than the direct shear test.

Apparatus

The triaxial machine consists of a pressure cell assembly and equipment for loading and measuring the load and deformation. The triaxial test set up is shown diagramatically in figure 5.1.

Triaxial pressure cell. Cylindrical soil specimen, inserted in a thin rubber membrane and kept sealed to prevent the entry of lateral fluid, can be placed in position in the cell. There is provision to apply radial fluid pressure and vertical stress through a piston. There is also facility to allow or prevent drainage of specimen during the application stresses. A pressure gauge may be used to measure the fluid pressure in the cell.

Loading equipment. Usually strain controlled equipment is used; stress controlled equipment is also used in some tests. The vertical load applied on the piston may be measured by a proving ring assembly. The deformation of specimen is measured by a dial dauge attached to the piston.

The triaxial test equipment may also have attachments to measure the volume changes of the specimen, and to note the pore water pressure in saturated specimens. Other components needed are sample trimmers moulds, porous discs, membrane, binding strips etc.

Procedure

Preparation of specimen. Either undisturbed or remoulded specimens may be prepared as desired. Larger size of undisturbed soil sample may be taken and trimmed to desired size. But undisturbed soil sample can be prepared only from soils having sufficient cohesion. Remoulded samples of cohesive soils may be prepared either by compacting first in a larger mould and by pressing hollow cutters or in constant volume moulds. Special care is needed while preparing specimens of cohesionless soil, in special mould after placing the membrane in proper position. The cylindrical specimens usually have height to diameter ratio equal to two; this ratio does not however exceed 2.5.

Test type. Three types of triaxial tests may be performed in partially or fully saturated specimens. These are *(i) undrained* or *quick* tests *(ii) consolidated — undrained* tests and *(iii) drained* or *slow* tests.

In the undrained or quick tests the outlet valve is closed and no drainage is allowed from the specimen during the test, from the time of application of lateral pressure σ_3 till the specimen fails under gradually increasing vertical load. In the consolidated undrained test, the drainage valve is kept open and the

specimen is allowed to fully consolidate under the applied lateral pressure σ_3; but no further drainage is allowed during the application of the vertical load, till failure.

In the drained or slow test the drainage is allowed during all stages of testing. First the specimen is allowed to fully consolidate under the applied lateral pressure σ_3 and later the vertical load is also applied in such a way that there is enough time for the drainage of pore water pressure developed from time to time.

There are two methods of applying the lateral or confining pressure. Usually the lateral pressure σ_3 is maintanied constant throughout the test. But in some studies the volume of the specimen is maintained constant by adjusting the lateral pressure.

Test procedure. The specimen enveloped properly in the membrane is kept in the triaxial cell and a desired lateral pressure σ_3 is applied. Then the vertical load is increased till the specimen fails noting the vertical deformation and load readings at desired intervals. The experiment is repeated for various other values of lateral pressure. To find the values of cohesion and angle of internal friction, tests should be carried out with atleast two or three different lateral pressure values. Soils may be tested with lateral pressures of 0, 0.75 and 1.5 kg/cm².

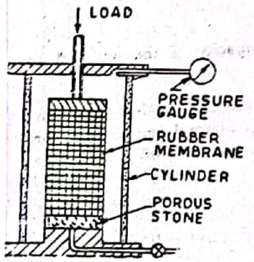

Figure 5.1
Triaxial Test Set-up

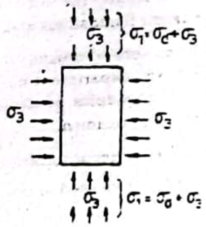

Figure 5.2
Stress Condition in Traxial Test

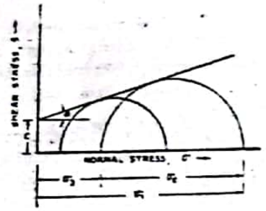

Figure 5.3
Mohr's Envelope from Triaxial Test

Calculations

The triaxial test specimen is subjected to the all round pressure equal to the lateral pressure σ_3 and the applied vertical stress or deviator stress σ_d such that the total vertical stress $\sigma_1 = \sigma_d + \sigma_3$. This is illustrated in Fig. 5.2. Mohr stress circles are polled at normal stress intercepts σ_3 and σ_1 or with diameters equal to deviator stresses. Mohr rupture envelope is then obtained by drawing a tangent to the circles, as shown in Fig. 5.3. The intercept of this line with Y—axis represents the cohesion C and the inclination with X– axis represents the angle of internal friction ϕ of the soil.

Correction for area of cross section. It is necessary to correct the deviator stress values for the increased area of cross section due to loading. Assuming that the volume of specimen remains constant and the area of cross section of the specimen increases uniformly, the corrected value of deviator stress σ_d is calculated from the relation :

$$\sigma_d = \frac{P_1}{A_c}\left(1 - \frac{\Delta}{l_o}\right)$$

Here P_1 = applied load

A_c = original area of cross section

Δ = deformation of specimen

l_o = original length of specimen.

Beside finding the values of C and ϕ, the behaviour of the soil is often judged from the stress-strain curves of the specimen. The value of *modulus of deformation*, E is also obtained from the stress-strain curve of the specimen obtained under specified test conditions, such as the value of lateral confinement, σ_3. The modulus of deformation is the ratio of stress to strain at an arbitrary point on the stress-strain curve; the point may be decided based on anticipated stress, or allowable percentage of strain.

Discussion

While testing subgrade soil for pavement design either an undrained or consolidated undrained test is preferred as the load applied is mostly *transient*.

The trixial test may also be carried out on soil aggregate mixtures, stabilized soils and bituminous mixtures. However larger size specimen should be used based on the maximum particle size in the test sample.

The unconfined compression test carried out on cohesive soils may be considered as a special case of the triaxial test when the lateral pressure σ_3 is equal to zero.

Applications of Triaxial Test Results

(a) The shear strength parameters C and ϕ of the materials may be used to find the shearing resistance of the material, using Coulomb's equation,

$$S = C + \sigma \tan \phi$$

These values are also useful to evaluate the ultimate bearing capacity of the soil and for evaluating the stability of embankment foundations and slopes.

(b) In the triaxial method of flexible pavement design, the E value of subgrade soils is to be found from the triaxial test. Kansas Highway department has recommended a lateral confining pressure, $\sigma_3 = 1.4$ kg/cm^2 during the test. The modulus of deformation of the subgrade soil is determined at axial strain value of 3.3% of the test specimen.

(c) Triaxial test is used in the design of bituminous mixes.

(d) The unconfined compression test ($\sigma_3 = 0$) is used to test bonded materials like soil-cement etc.

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2. Soil Testing for Engineers, *T.W. Lambe*, John Wiley and sons, New York.
3. Principles of Pavement Design, *E. J. Yoder*, John Wiley and sons, New York.
4. Methods of Tests for Soils, Part XI, *Indian Standards Insititutions*.

PROBLEMS

1. Compare trixial and direct shear tests.
2. Briefly explain the principle of triaxial test.
3. Write a note on preparation of triaxial specimens.
4. How are C and ϕ values found from triaxial test results.
5. Why is correction of stress necessary for change in area of cross section ? How is the correction made and why ?
6. Define modulus of deformation. What are the applications ?
7. What are the various applications of triaxial test results ?

OBSERVATION SHEET : TRIAXIAL TEST

Soil type : Type of test : undrained/consolidated undrained/drained

Moisture content = % Size of specimen =

Proving ring calibration factor = Initial area, A_0 =

Deformation dial, 1 division = cm Initial length, l_0 =

Lateral pressure σ_3, kg/cm^2	Deformation dial readings	Proving ring dial reading	Deformation Δ, cm	Vertical load P, kg	Deviator stress $\sigma_d = \frac{P}{A_0}\left(1 - \frac{\Delta}{l_0}\right)$	Volume change reading, cm^3	Pore pressure readings kg/cm^2	Remarks

6

California Bearing Ratio Test

INTRODUCTION

The California Bearing Ratio (CBR) test was developed by the California Division of Higway as a method of classifying and evaluating soil-subgrade and base course materials for flexible pavements. Just after World War II, the U.S. Corps of Engineers adopted the CBR test for use in designing base course for airfield pavements. The test is empirical and results can not be related accurately with any fundamental property of the material. The method of test has been standardised by the ISI also.

The CBR is a measure of resistance of a material to penetration of standard plunger under controlled density and moisture conditions. The test procedure should be strictly adhered if high degree of reproducibility is desired. The CBR test may be conducted in re-moulded or undisturbed specimens in the laboratory. U. S. Corps of Engineers have also recommended a test procedure for in-situ test. Many methods exist today which utilise mainly CBR test values for designing pavement structure. The test is simple and has been extensively investigated for field correlations of flexible pavement thickness requirement.

Briefly, the test consists of causing a cylindrical plunger of 50 mm diameter to penetrate a pavement component material at 1.25 mm/minute. The loads, for 2.5 mm and 5 mm are recorded. This load is expressed as a percentage of standard load value at a respective deformation level to obtain CBR value. The standard load values were obtained from the average of a large number of tests on different crushed stones and are given in Table 6.1.

Table 6.1 Standard Load Values on Crushed Stones for Different Penetration Values

Penetration, mm	Standard Load, kg.	Unit standard load, kg/cm^2
2.5	1370	70
5.0	2055	105
7.5	2630	134
10.0	3180	162
12.5	3600	183

LABORATORY C.B.R. TEST

Apparatus

(a) Loading Machine : Any compression machine which can operate at a constant rate of 1.25 mm per minute can be used for this purpose. If such machine is not available then a calibrated hydraulic press with proving ring to measure load can be used. See Figure 6.1. A metal penetration piston or plunger of diameter 50 mm is attached to the loading machine.

(b) Cylinderical Moulds : Moulds of 150 mm diameter and 175 mm height provided with a collar of about 50 mm length and detachable perforated base are used for this purpose. A spacer disc of 148 mm

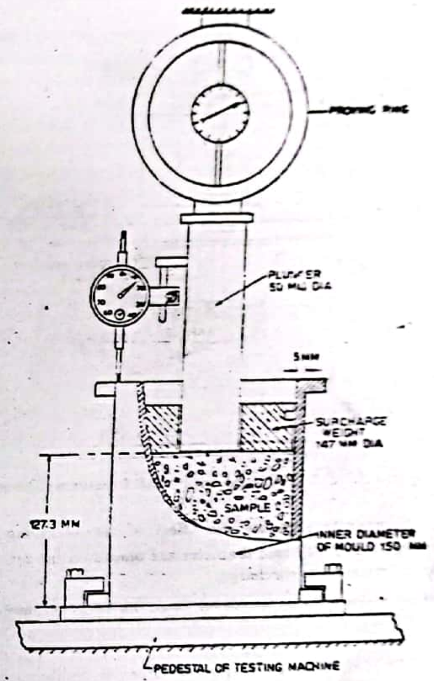

Figure 6.1 C.B.R. Test Apparatus

diameter and 47.7 mm thickness is used to obtain a specimen of exactly 127.3 mm height.

(c) Compaction Rammer : The material is usually compacted as specified for the work, either by dynamic compaction or by static compaction. The details for dynamic compaction suggested by the ISI are given in Table 6.2.

Table 6.2 Specifications for Dynamic Compaction

Type of compaction	Number of layers	Magnitude of blows: Weight of hammer, kg	Fall, cm	Number of blows
Light Compaction	3	2.6	31	56
Heavy Compaction	5	4.89	45	56

(d) Adjustable stem, perforated plate, tripod and dial gauge : The standard procedure requires that the soil sample before testing should be soaked in water to measure swelling. For this purpose the above listed accessories are required. See the arrangement in Figure 6.2.

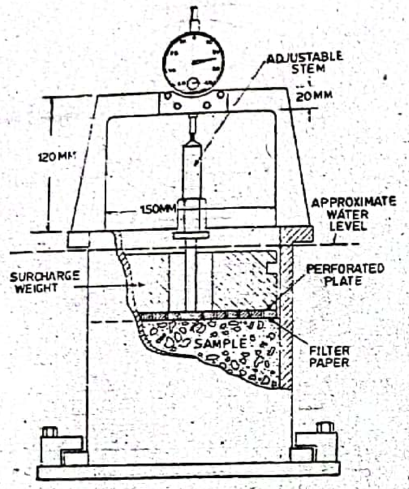

Figure 6.2 C.B.R. Mould with Swell Measuring Device

(e) *Annular weight* : In order to simulate the effect of the overlying pavement weight, annular weights each of 2.5 kg weight and 147 mm diameter are placed on the top of the specimen, both at the time of soaking and testing the samples, as surcharge.

Besides above equipment, coarse filter paper, sieves, oven, balance, etc. are required.

Procedure

As per the ISI, the CBR test may be performed either on undisturbed soil specimens obtained by fitting a cutting edge to the mould or on remoulded specimens. Remoulded soil specimens may be compacted either by static compaction or by dynamic compaction. When static compaction is adopted, the batch of soil is mixed with water to give the required moisture content; the correct weight of moist soil to obtain the desired density is placed in the mould and compaction is attained by pressing in the spacer disc using a compaction machine or jack. The preparation of soil specimens by dynamic compaction or ramming is more commonly adopted and is explained below.

About 45 kg of material is dried and sieved through 20 mm sieve. If there is note worthy proportion of materials retained on 20 mm sieve, allowance for larger size materials is made by replacing it by an equal weight of material passing 20 mm sieve and retained on 4.75 mm sieve. The optimum moisture content and maximum dry density of the soil are determined by adopting either IS light compaction (Proctor compaction) or IS heavy compaction (modified Proctor or modified AASHO compaction) as per the requirement.

Each batch of soil (of atleast 5.5 kg weight for granular soils and 4.5 to 5.0 kg weight for fine grained soils) is mixed with water upto the optimum moisture content or the field moisture content if specified so. The spacer disc is placed at the bottom of the mould over the base plate and a coarse filter paper is placed over the spacer disc. The moist soil sample is to be compacted over this in the mould by adopting either the IS light compaction or the IS heavy compaction.

(i) For IS light compaction or Proctor compaction the soil to be compacted is divided into three equal parts; the soil is compacted in three equal layers, each of compacted thickness about 44 mm by applying 56 evenly distributed blows of the 2.6 kg rammer.

(ii) For IS heavy compaction or the modified Proctor compaction, the soil is divided into five equal parts; the soil is compacted in five equal layers, each of compacted thickness about 26.5 mm by applying 56 evenly distributed blows of the 4.89 kg rammer. After compacting the last layer, the collar is removed and the excess soil above the top of the mould is evenly trimmed off by means of the straight edge. It is important to see if the excess soil to be trimmed off while preparing each specimen is of thickness about 5.0 mm; if not the weight of soil taken for compacting each specimen is suitably adjusted for the repeat tests so that the thickness of the excess layer to be trimmed off is about 5.0 mm. Any hole that develops on the surface due to the removal of coarse particles during trimming, may be patched with smaller size material. Three such compacted specimens are prepared for the CBR test. About 100 g of soil samples are collected from each mould for moisture content determination, from the trimmed off portion.

The clamps are removed and the mould with the compacted soil is lifted leaving below the perforated base plate and the spacer disc which is removed. The mould with the compacted soil is weighed. A filter paper is placed on the perforated base plate, the mould with compacted soil is inverted and placed in position over the base plate (such that the top of the soil sample is now placed over the base plate) and the clamps of the base plate are tightened. Another filter paper is placed on the top surface of the sample and the perforated plate with adjustable stem is placed over it. Surcharge weights of 2.5 or 5.0 kg weight are placed over the perforated plate and the whole mould with the weights is placed in a water tank for soaking such that water can enter the specimen both from the top and bottom. The swell measuring device consisting of the tripod and the dial gauge are placed on the top edge of the mould and the spindle of the dial gauge is placed touching the top of the adjustable stem of the perforated plate. (See Fig. 6.2). The initial dial guage reading is recorded and the test set up is kept undisturbed in the water tank to allow soaking of the soil specimen for four full days or 96 hours. The final dial gauge reading is noted to measure the expansion or swelling of the soil specimen due to soaking.

The swell measuring assembly is removed, the mould is taken out of the water tank and the sample is allowed to drain in a vertical position for 15 minutes. The surcharge weights, the perforated plate with stem and the filter paper are removed. The mould with the soil sample is removed from the base plate and is weighed again to determine the weight of water absorbed.

The mould with the specimen is clamped over the base plate and the same surcharge weights are placed on the specimen centrally such that the penetration test could be conducted. The mould with base plate is placed under the penetration plunger of the loading machine. The penetration plunger is seated at the centre of the specimen and is brought in contact with the top surface of the soil sample by applying a seating load of 4.0 kg. The dial gauge for measuring the penetration values of the plunger is fitted in position. The dial gauge of the proving ring (for load readings) and the penetration dial gauge are set to zero. The load is applied through the penetration plunger at a uniform rate of 1.25 mm/min. The load readings are recorded at penetration readings of 0.0, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0, 7.5, 10.0 and 12.5 mm. In case the load readings start decreasing before 12.5 mm penetration, the maximum load value and the corresponding penetration value are recorded. After the final reading, the load is released and the mould is removed from the loading machine. The proving ring calibration factor is noted so that the load dial values can be converted into load in kg. About 50 g of soil is collected from the top three cm depth of the soil sample for the determination of moisture content.

Calculation

The swelling or expansion ratio is calculated from the observations during the swelling test using the formula :

$$\text{Expansion ratio} = \frac{100\,(d_f - d_i)}{h}$$

where d_f = final dial gauge reading after soaking, mm

d_i = initial dial gauge reading before soaking, mm

h = initial height of the specimen, mm.

The load values noted for each penetration level are divided by the area of the loading plunger ($19.635\ cm^2$) to obtain the pressure or unit load values on the loading plunger. The load - penetration curve is then plotted in natural scale for each specimen as shown in Fig. 6.3. If the curve is uniformly convex upwards as shown for specimen no. 1, no correction is needed. In case there is a reverse curve or the initial portion of the curve is concave upwards as shown for specimen no. 2, necessity of a correction is indicated. A tangent is drawn from the steepest point on the curve (point X in Fig. 6.3) to intersect the base at point Y which is the corrected origin corresponding to zero penetration. The unit load values corresponding to 2.5 and 5.0 mm penetration values (either from the original origin for curve without correction or from the corrected origin for the curve with correction, as the case may be) are found from the graph. The CBR value is calculated from the formula :

$$\text{CBR, percent} = \left[\frac{\text{Unit load carried by soil sample at defined penetration level}}{\text{Unit load carried by standard crushed stones at above penetration level}}\right] \times 100$$

The unit load values on standard crushed stones are given in Table 6.1.

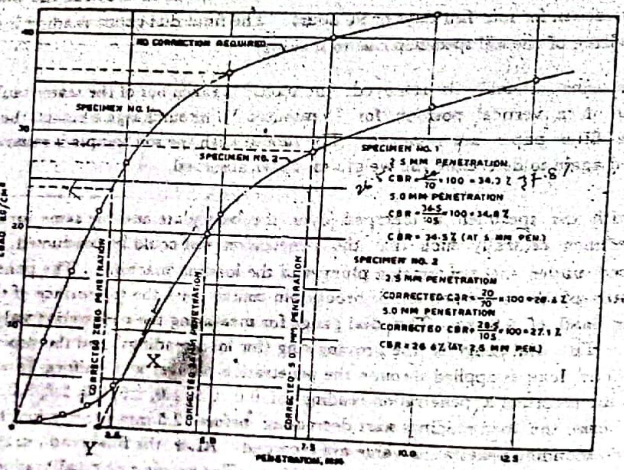

Figure 6.3 C. B. R. Test Load-Penetration Curve

Results

The expansion ratio of soil due to soaking and the other details of the test may be reported as given in the observation sheet.

The CBR values at 2.5 mm and 5.0 mm penetrations are calculated for each specimen from the corresponding graphs. Generally the CBR value at 2.5 mm penetration is higher and this value is adopted. However if higher CBR value is obtained at 5.0 mm penetration, the test is to be repeated to verify the results; if the value at 5.0 mm is again higher, this is adopted as the CBR value of the soil sample. The average CBR value of three specimens is reported to the first decimal place.

According to the Indian Roads Congress, if the maximum variation in laboratory in CBR values between the three specimens exceeds the values given below for the different ranges, the CBR tests should be repeated on additional three specimens and the average value of six specimens is adopted.

Maximum permissible variation in CBR values, %	Range of CBR values, %
3.0	upto 10
5.0	10 to 30
10.0	30 to 60
Not significant	above 60

Discussion

Undisturbed soil samples may be used for the CBR test by taking out samples from the field in the mould by attaching a core cutter. Due to high degree of disturbance in sample, this method is generally not adopted.

The CBR test is essentially an arbitrary strength test and hence can not be used to evaluate the fundamental soil properties. Unless the test procedure is strictly followed, dependable results cannot be obtained. The compaction specifications such as total height of compacted specimen (before trimming off), the equality of thickness of the five compacted layers and the uniformity of distribution the blows of the rammer in each layer (in the case of dynamic compaction) affect the test results. The initial upward concavity of the load-penetration curve calling for the correction may be due to (i) piston surface not being fully in contact with top of the specimen or (ii) the top-layer of soaked soil being too soft. The test is meant only for soil and granular base course materials and hence may not be suitable for semi-rigid materials like soil-cement.

5.2 FIELD C.B.R. TEST

Apparatus

A reaction load like a truck, tractor or truss is required for applying the load by means of a mechanical screw jack. The other equipment needed are 5 cm diameter loading plunger, extentsion rods, jacks, proving ring assembly, dial gauge, datum frame, annular surcharge plate 25 cm in diameter and 5 kg in weight, with a central hole and slot width 5.3 cm and two circular slotted weights of 10 kg and diameter about 25 cm with central hole and slot width of 5.3 cm.

Procedure

A circular area of about 30 cm in diameter is trimmed and levelled. Particular care should be taken at the centre where the plunger is to be seated. The surcharge load of 15 kg is placed on this surface and the plunger is seated properly. The dial gauge to measure the penetration is attached to the plunger from an independent datum frame. A seating load of 4 kg is applied and the load and penetration dials are set to zero.

The load is applied to the plunger by means of the jack such that the rate of penetration is approximately 1.25 mm/minute. The load readings are noted for at penetrations 0.0, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0, 7.5, 10.0 and 12.5 mm. The load is released and moisture content specimen is taken from underneath the plunger.

Calculation

The load-penetration curve is plotted, and the C.B.R. value is calculated as in the case of laboratory C.B.R. The correction is applied where necessary i.e., in the case of load-penetration curves which are concave initially upwards.

Three in-place CBR tests shall be performed at each elevation to be tested and the average value is adopted. However if the three tests in any group do not show reasonable agreement within the maximum specified variation for the range of CBR values, (tolerance of 3 for conducted CBR values upto 10%, 5 for values 10 to 30%, 10 for 30 to 60% and 25 for values greater than 60%) three additional tests shall be made in the average of six tests be adopted for design.

Discussion

The field-CBR value is found at the existing moisture content. It is not easy to obtain the field CBR value under soaked condition, unless the test site is subjected to soaking by flooding prior to the test. It is not possible to satisfactorily simulate the critical conditions of dry density and moisture content in the field. As the CBR value of a soil largely depends on the density and moisture content of the soil, the test conditions should be selected with due care.

Applications of CBR test

Based on extensive CBR test data collected, empirical design charts were developed by the California

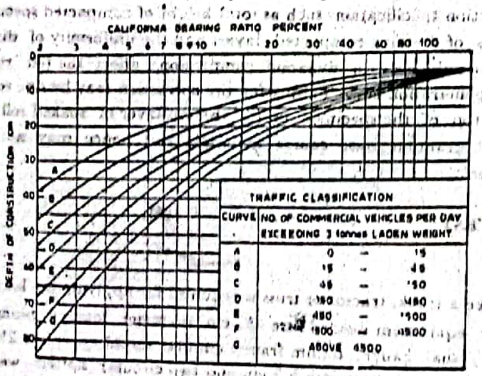

Figure 6.4 CBR Design Chart

State Highway Department, correlating the CBR value and flexible pavement thickness requirements. Later on similar design curves were prepared by various other agencies etc. The CBR design chart adopted by the Indian Roads Congress for tentative use in India is given in Figure 6.4. The different curves are for the different traffic volumes.

As an example, if the CBR value of a subgrade soil is 7 percent and the traffic volume expected is between 450 and 1500 commercial vehicles per day, using curve E (see Figure 6.4) the total flexible pavement thickness is obtained as 35 cm.

REFERENCES

1. Methods of Test for Soils Part XVI, Laboratory Determination of CBR, IS : 2720, *Indian Standards Institution.*
2. Soil Mechanics for Road Engineers, *DSIR*, H.M.S.O., London.
3. Highway Engineering, *S.K. Khanna*, and *C.E.G. Justo*, Nem Chand & Bros, Roorkee.
4. Principles of Pavement Design, *E.J. Yoder*, John Wiley and sons, New York.
5. Guidelines for the Design of Flexible Pavements, I.R.C. : 37—1970, *Indian Roads Congress.*
6. Methods of Test for Soils, Part XXXI Field Determination of California Bearing Ratio, IS : 2720, *Indian Standards Institution.*

PROBLEMS

1. Explain CBR.
2. What is the significance of surcharge load while soaking and testing the soil ?
3. Discuss the objects of soaking CBR sample before testing.
4. How is CBR value evaluated from the graph ?
5. When is correction of CBR curve called for ?
6. What are the causes for the initial concavity of the CBR curve ?
7. CBR values of a soil obtained at 2.5 mm and 5.0 mm penetration are 5 and 6.3 percent respectively, even after repeated testing. What CBR values should be adopted for design ?
8. CBR value of soil A is 15 and of soil B is 4. Which one is a better soil ? Why ?
9. Briefly outline the CBR test procedure.
10. What are the applications of CBR test.
11. Calculate the total pavement thickness over a soil subgrade of CBR value 6 percent. Assume traffic of about 300 commercial vehicles per day.
12. Discuss the limitations of CBR test.
13. How is field CBR test carried out ?

Procedure

A circular area of about 30 cm in diameter is trimmed and levelled. Particular care should be taken at the centre where the plunger is to be seated. The surcharge load of 15 kg is placed on this surface and the plunger is seated properly. The dial gauge to measure the penetration is attached to the plunger from an independent datum frame. A seating load of 4 kg is applied and the load and penetration dials are set to zero.

The load is applied to the plunger by means of the jack such that the rate of penetration is approximately 1.25 mm/minute. The load readings are noted for at penetrations 0.0, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0, 7.5, 10.0 and 12.5 mm. The load is released and moisture content specimen is taken from underneath the plunger.

Calculation

The load-penetration curve is plotted, and the C.B.R. value is calculated as in the case of laboratory C.B.R. The correction is applied where necessary i.e., in the case of load-penetration curves which are concave initially upwards.

Three in-place CBR tests shall be performed at each elevation to be tested and the average value is adopted. However if the three tests in any group do not show reasonable agreement within the maximum specified variation for the range of CBR values, (tolerance of 3 for conducted CBR values upto 10%, 5 for values 10 to 30%, 10 for 30 to 60% and 25 for values greater than 60%) three additional tests shall be made in the average of six tests be adopted for design.

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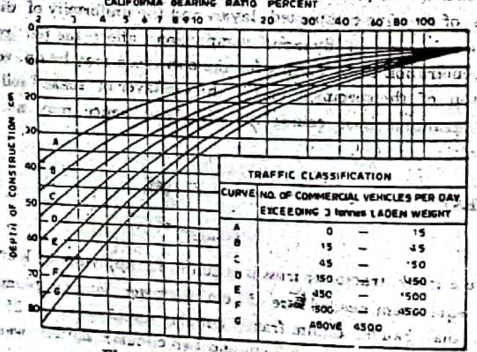

Figure 6.4 CBR Design Chart

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PROBLEMS

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OBSERVATION SHEET

CALIFORNIA BEARING RATIO TEST

Compacting moisture content =

Dry Density =

Condition of test specimen :—soaked/unsoaked

Moisture content (a) At top 3 cm layer after soaking =

(b) Average after soaking =

Proving ring calibration factor =

Surcharge weight = Period of soaking = Expansion ratio =

Sample Number (1)	Penetration mm (2)	Proving ring dial reading (3)	Load on Plunger, kg (4)	Corrected load, kg (5)	Unit load, kg/cm² (6)
	0.00				
	0.5				
	1.0				
	1.5				
	2.0				
	2.5				
	3.0				
	4.0				
	5.0				
	7.5				
	10.0				
	12.5				

CBR at 2.5 CBR at 5.0 mm

1	2	3	4	5	6

CBR at 2.5 mm...... ......CBR at 5.0 mm......

1	2	3	4	5	6

CBR at 2.5 mm.............. CBR at 5.0 mm...... ....

Average CBR value at penetration mm = %

Remarks

North Dakota Cone Test

INTRODUCTION

The North Dakota Cone (NDC) Test is an ad hoc penetration test developed by North Dakota State Highway Department of the U.S.A. for use in flexible pavement design. This cone penetration test method can be used to test in-situ soils in the field as well as in the laboratory to test soils remoulded in large moulds such as the CBR mould. The main advantage of this test is that the equipment being portable and simple, can easily be used in field control tests of soil, soil-bitumen etc., when the soil is free from coarse particles. The NDC test has been standardised by the ISI.

Apparatus

The North Dakota Cone Test apparatus consists of a vertical shaft with a sharp steel cone of angle 15° 30' attached to one end, which could move relative to its supporting frame and is provided with a locking device. See Figure 7.1. The Cone is loaded by placing weights on the top plate and the penetration of the cone is measured by a graduated scale. The moving part connected to the cone assembly and top plate weighs 5 kg. A set of 7 weights each 5 kg is used for loading and a stop watch is used for noting the loading time.

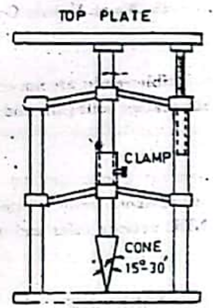

Figure 7.1 North Dakota Cone Test Apparatus

Procedure

When the test is to be conducted on the subgrade or in-situ soil, the area is first trimmed level. The apparatus is placed in position and the cone is moved down until the tip just touches the surface of the soil. The shaft is locked in this position and the initial reading of the penetration scale is noted. The shaft is then unlocked and simultaneously the stop watch is started. The penetration is allowed for one minute, the shaft is locked and the penetration scale reading is noted. The difference between these two penetration readings gives the penetration of the cone under a load of 5 kg, being the weight of moving cone assembly.

The load is next increased to 10 kg by placing a 5 kg weight on the top plate and the penetration is noted by unlocking the shaft for one minute. This procedure is repeated for total loads of 20 and 40 kg (including the weight of the cone assembly).

The NDC test may also be conducted on compacted (and soaked) specimens prepared in the CBR mould, following the same test procedure as above.

Calculations

Though theoretically the bearing pressure or the load on the code divided by the cross sectional area of the penetrated cone at the soil surface level should be equal for each of the applied loads, in practice this is not so because of zero error due to the blunt point of the cone. If the cone has a true point, for equal bearing pressure, the penetration at 10 kg load should be half that for 40 kg load. Therefore, if the correction 'C' due to the rounded point is given by :

$$C = p_{40} - 2p_{10}$$

where p_{10} and p_{40} are the penetration values corresponding to 10 and 40 kg loads respectively. This correction C is added to all the observed penetration readings to get corrected penetration readings.

As the cone angle is 15° 30′, the bearing area for a corrected penetration value of p_c is equal to $\pi p_c^2 \tan^2 (7° 45') = 0.058\ p_c^2$. The bearing value for each load is calculated by dividing the load by the corresponding bearing area.

Results

The results are tabulated as shown in the observation sheet and the average bearing value, excluding the first reading with 5 kg total load, is taken as the North Dakota Cone Bearing value of the soil.

Discussion

If the soil contains coarse particles, reproducible results are not obtained by the test. Therefore, the use of North Dakota Cone test is limited to fine grained soils (silts and clays) free from coarse particles.

Applications of NDC Test

This test is made use of in flexible pavement design method evolved by the North Dakota State Highway Department and is known as North Dakota Cone Method of Design. An empirical design chart as well as an equation correlating the NDC bearing value and pavement thickness requirement have been made available.

REFERENCES

1. Soil Mechanics for Road Engineers, *DSIR*, H.M.S.O., London.
2. Methods of Test for Soils, Part XXXII North Dakota Cone Lest, IS : 2720, *Indian Standards Institution*.
3. Principles of Pavement Design, *E. J. Yoder*, John Wiley & Sons.

PROBLEMS

1. What is the principle of North Dakota Cone test.
2. Why is correction made to the penetration readings of the NDC test ? How is this correction calculated ?
3. How is the NDC bearing value determined ?
4. What are the limitations of this test ?
5. What are the uses and applications of the NDC test ?

OBSERVATION SHEET

Soil type :

Test type : In-situ/remoulded

Correction $C = p_{40} - 2p_{10}$

Load, kg	Penetration reading, p, mm	Corrected penetration, p_c, cm	Bearing value kg/cm²
0			
5			
10			
20			
40			

Mean NDC Bearing Value =

8

Plate Bearing Test

INTRODUCTION

In plate bearing test, a compressive stress is applied to the soil or pavement layer through rigid plates of relatively large size and the deflections are measured for various stress values. The deflection level is generally limited to a low value, in the order of 1.25 to 5 mm and so the deformation caused may be partly elastic and partly due to compaction of the stressed mass with negligible plastic deformation. The plate bearing test has been devised to evaluate the supporting power of subgrades or any other pavement layer by using plates of large diameter.

The plate bearing test was originally meant to find the *modulus of subgrade reaction* in the Westergaard's analysis for wheel load stresses in cement concrete pavements. The procedure for determining the Westergaard's modulus of subgrade reaction, 'K' is given here. Various organisations have specified standard test procedures.

Apparatus

The apparatus consists of bearing plates, loading equipment and instruments to measure the applied loads and resulting settlement.

(a) *Bearing plates* : consist of a mild steel 75 cm in diameter and 1.5 to 2.5 cm thickness, and few other plates of same thickness, but smaller diametres (usually 60, 45, 30, and 22.5 cm dia.) used as stiffners.

(b) *Loading equipment* : consist of a reaction or dead load and a hydraulic jack. The reaction frame may suitably be loaded to give the needed reaction load on the plate. The load applied may be measured either by a proving ring and dial gauge assembly or a pressure gauge connected to the output end of the hydraulic jack.

(c) *Settlement measurements* : may be made by means of three or four dial gauges fixed on the perifery of the bearing plate from an independent datum frame. The datum frame should be supported far from the loaded area.

The plate bearing test set-up is shown in Figure 8.1.

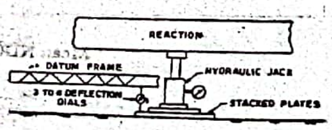

Figure 8.1 Plate Bearing Test Set-up

Procedure

The test site is prepared and loose material is removed so that the 75 cm diameter plate rests horizontally in full contact with the soil sub-grade. If the modulus of sub-grade reaction of natural ground is needed, the top soil may be removed up to a depth of about 20 cm before testing.

The plate is seated accurately and then a seating load equivalent to a pressure of 0.07 kg/cm² (320 kg for 75 cm diameter plate) is applied and released after a few seconds. The settlement dial readings are now noted corresponding to zero load. A load is applied by means of the jack, sufficient to cause an average settlement of about 0.25 mm. When there is no perceptible increase in settlement or when the rate of settlement is less than 0.025 mm per minute (in the case of soils with high moisture content or in clayey soils) the load dial reading and the settlement dial readings are noted. The average of the three (or four) settlement dial readings is taken as the average settlement of the plate corresponding to the applied load.

The load is then increased till the average settlement increase to a further amount of about 0.25 mm, and the load and average settlement readings are noted as before. The procedure is repeated till the settlement is about 1.75 mm or more.

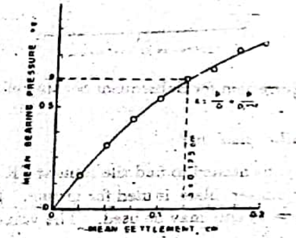

Fig. 8.2 Bearing Pressure-Settlement Curve

Calculation

A graph is plotted with the mean settlement versus bearing pressure (load per unit area) as shown in Figure 8.2. The pressure p (kg/cm²) corresponding to a settlement $\Delta = 0.125$ cm is obtained from this graph. The modulus of subgrade reaction K is calculated from the relation

$$K = \frac{p}{0.125} \text{ kg/cm}^2\text{/cm or kg/cm}^3$$

Discussion

(i) *Correction in K-value for subsequent soaking of subgrade*

The modulus of subgrade reaction K of a soil found by this method would depend on the moisture content of the soil at the time of testing. The minimum value of K will be obtained under soaking conditions of the subgrade. But it may not be often practicable to carryout this test at such soaked condition. If the value of K is desired for the soaked conditions of soil, then a correction factor may be used to multiply the K value found at the prevailing field moisture content.

Two soil samples are collected from the site where plate load test was conducted, at the field moisture content. One of these samples is subjected to consolidation test at field moisture content (unsoaked test). The other specimen is subjected to the consolidation test after saturating it by soaking in water. Two curves are plotted with the deformation or settlement values on the X-axis and the pressure values on the Y-axis as shown in Fig. 8.3. The pressure, p obtained in the plate load test is fed into the graph for unsoaked specimen and the pressure value, p_s of soaked specimen to cause the corresponding deformation value is determined. The correction factor for soaking is taken as $\frac{p_s}{p}$, which will be less than 1.0. Therefore the modulus of subgrade reaction under soaked condition, K_s is obtained using the relation :

$$K_s = K \frac{p_s}{p}$$

where, K is the modulus of subgrade reaction determined at field moisture content.

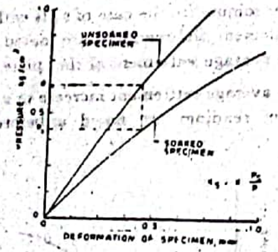

Fig. 8.3 Correction for Subsequent Soaking of Subgrade

(ii) Correction in K-value for smaller plate size

Very heavy reaction load may be needed to find the standard K-value of soils with higher bearing capacity when standard, 75 cm diameter plate is used for testing. If the reaction load available is less, a plate of smaller diameter ($2 a_1 = 30$ cm) may be used. The value of subgrade modulus K_1 obtained with smaller plate should be corrected for the standard diameter of 75 cm, assuming that K.a is a constant.

$$\text{ie., } K = \frac{K_1 a_1}{a}$$

Applications of the Plate Bearing Test

The modulus of subgrade reaction K is used in rigid pavement analysis for determining the radius of relative stiffness l, using the relation :

$$l = \left[\frac{E h^3}{12 K (1-\mu^2)}\right]^{\frac{1}{4}}$$

Apart from the need to find K value, the plate bearing test has the following uses in pavement design and evaluation :

(i) The exact load-deflection behaviour of the soil or the pavement layer in-situ for static loads is obtained by the plate bearing test. The loaded area may be kept equal to the actual area in field. The supporting power of the soil subgrade or a pavement layer may be found in pavement evaluation work.

(ii) Repeated plate bearing test is carried out to find the subgrade support in flexible pavement design by Mc Leod method.

(iii) The elastic modulus values and the ratio of E_1/E_2 are found by carrying out plate load tests on the subgrade and the base coarse layer, in flexible pavement design using Burmister's elastic two-layer theory.

REFERENCES

1. Soil Mechanics for Road Engineers *D.S.I.R.*, H.M.S.O., London.
2. Principles of Pavement Design, *E.J. Yoder*, John Wiley and Sons, New York.
3. Highway Engineering *S.K. Khanna* and *C.E.G. Justo*, Nem Chand & Bros., Roorkee.

PROBLEMS

1. Define Modulus of subgrade reaction. Discuss the application.
2. Describe briefly the procedure for finding modulus of subgrade reaction.
3. The K value found using a 30 cm diameter plate is 10 kg/cm³. What is the K-value for a standard plate diameter of 7.5 cm.
4. What are the various uses of plate bearing test ?
5. How is the K value determined at field moisture content, corrected for subsequent soaking condition ?
6. What is the objective of conducting plate bearing test using smaller size plate ? How is the correction in K value made for non-standard plate size used in the test ?

OBSERVATION SHEET
PLATE BEARING TEST

Soil type : 　　　　　　　　Plate diameter =

Moisture content = %　　　　Area of plate =

Settlement dial, 1 division = mm　　Proving ring calibration factor =

Approximate settlement, mm	Settlement dial readings, divisions 1	2	3	4	Average settlement, Δ mm	Load dial (proving ring dial) reading, divisions	Load per unit area, p kg/cm²	Remarks
0.00								
0.25								
0.50								
0.75								
1.00								
1.25								
1.50								
1.75								

Modulus of subgrade $K = \frac{p}{\Delta} = \frac{p}{0.125}$ kg/cm³

(p = the pressure corresponding to average settlement Δ = 0.125 cm, obtained from the graph Δ Vs. p-values)

9

Soil-Cement Mix Design

INTRODUCTION

Soil-cement is an intimate mixture of pulverized soil, cement and water compacted and cured for the desired period producing a hardened and stable mass. Soil-cement has been mainly used as sub-base and base course material for highway and airfield pavements. For all soils the strength of soil-cement increases with cement content. It has been a controversial matter to decide the basis for the design of a soil-cement mix ; the points generally considered are :

(i) desired strength

(ii) resistance to adverse weather conditions including alternate wet-dry and/or freeze-thaw cycles.

(iii) Volume and moisture changes during the above weathering cycle.

Different mix design methods have been specified by different agencies for determining the cement content requirement for a soil-cement mix satisfactory in the field performance.

Many factors affect the properties and performance of soil-cement. They are :

(i) *Soil Type*—(a) Physical properties of soil including grain size distribution, plasticity characteristics (b) clay content and type of clay (c) deterimental matter in soil like organic matter and sulphates.

(ii) *Cement*—(a) Type of cement (b) Cement content

(iii) *Pulverisation and mixing*—(a) degree of pulverisation (b) mixing efficiency (c) mixing period

(iv) *Compaction*—(a) Type of compaction (b) amount of compaction (c) Compacting moisture content

(v) *Curing*—(a) Type of curing (b) curing temperature (c) period of curing.

Design, properties and performance of soil-cement mixes are complex and hence need thorough consideration.

Most well known methods for the design of soil-cement mixes are :

(i) *British Test Method :* This method specifies that the designed soil-cement mix should possess a minimum compressive strength of 17.5 kg/cm² at 7 days curing.

(ii) *Portland Cement Association (PCA) Design Method :* This method specifies some tests to estimate the resistance to adverse weathering conditions. Accelerated weathering tests have been specified for laboratory testing. The specimens are subjected to the prescribed wet-dry and freeze-thaw cycles for studing the resistance to disintegration in terms of brushing loss. Variation in volume and moisture conditions of the specimens are also recorded. PCA design method does not include any strength test for the mix design. ASTM has standardised the wet-dry and Freeze-thaw tests prescribed by the PCA.

(iii) *British Test method for Determination of Resistance to Immersion in Water :* In this method the compressive strength of specimens of size, 5 cm diameter × 10 cm height is determined after 7 days curing as well as after subjecting the specimens to prolonged soaking. The index of resistance to softening due to immersion in water is determined for the design of mix.

Indian Standard Methods of Test

Methods of test for stabilized soil have been standardised by the Indian Standards Institution.

mination of Unconfined Compressive Strength of soil-cement standardised by the ISI similar to the British test method. The Wetting and Drying and Freezing and Thawing tests for compacted soil-cement mixtures standardised by the ISI are similar to the PCA method and the ASTM standard methods of testing soil-cement. The ISI has also standardised the Flexural Strength of soil-cement using simple beam with third point loading.

DETERMINATION OF UNCONFINED COMPRESSIVE STRENGTH OF SOIL CEMENT

Apparatus

(a) Cylindrical moulds for preparing specimens of the specified sizes :

(i) 5 cm diameter × 10 cm height for fine grained soils containing not less than 90 percent particles passing 2.36 mm sieve.

(ii) 10 cm diameter × 20 cm height for medium grained soils containing not less than 90 percent of particles passing 20 mm sieve.

(b) Compaction apparatus consisting of 10 cm diameter and 1000 cc capacity mould and the hammer of weight 2.6 kg and height of fall of 31 cm and steel tamping rod.

(c) Compression testing machine capacity 5 tonnes having a rate of strain of 1.25 mm/minute and proving ring.

(d) Other equipment like straight edge, for moisture content determination, Jacks for compacting specimens in constant volume mould and for extruding the specimen. Arrangement for curing the specimen is also required. Paraffin wax or other suitable wax for coating specimen for curing or curing tin.

Procedure

This first step is to find the maximum dry density and optimum moisture content of soil-cement mixes. Hence suitable proportions of cement to be added are decided based on the soil type. For example, for a particular soil, the approximate percentage by weight of cement required to give the desired strength, estimated based on the classification group of the soil, is say, 10% by weight of soil. So the percentages of cement chosen for laboratory testing may be 5, 8, 10, 12, and 15. For each of the chosen percentage of cement, the maximum dry density and optimum moisture content of the soil-cement mix are determined by using IS light compaction method. Cement is added to the dry soil, mixed and then the required amount of water is added and wet mixing is done. The compaction test is carried out as explained in Experiment 3, *Compaction test*. Maximum dry density and optimum moisture content for the soil-cement mixes are determined.

The appropriate size of the mould for preparing specimens is selected based on the soil, as discussed above. Weight of the dry soil and cement needed for preparing the specimen at the maximum dry density is calculated. Soil sample is weighed out to prepare a batch of five or ten specimens. To this water is added to bring the water content to a value of 3 percent below the respective O.M.C. The wet soil is mixed and this may be stored in an air tight container for 24 hours to enable the moisture to get uniformly distributed throughout the soil.

The required amount of cement, calculated as a percentage of the dry weight of soil is then mixed by the laboratory mixer for about 1 to 2 minutes, and the remaining 3% water is added and mixing continued for a further 8—9 minutes. The soil-cement-water mixture prepared for 10 specimens are divided into ten equal parts using a balance and they are moulded in the constant volume moulds by placing the piston plugs on either ends. Compaction is done under static load applied through a compression testing machine or a jack and the pressure is maintained for about half a minute. The specimen may also be compacted

by driving home the end plugs with a hammer. The specimen is demoulded by removing the plugs and inserting the plunger into the end of the mould having the smaller diameter and by passing until the specimen is released. The specimen may be demoulded immediately in the case of most of the cohesive soils, but in sandy soils demoulding may preferably be done after a few hours (or after 24 hours) to enable some hardening of the soil-cement.

Alternatively specimens may be compacted with the constant compactive effort by taking sufficient quantity of soil-cement mix to give a compacted length of specimen of 10.0 to 11.5 cm in the case of fine grained soil and 20.0 to 21.5 cm in the case of medium grained soils. The plug at the top is then inserted and the fine grained soil-cement specimen is compacted by 15 blows of the rammer dropped freely from a height of 31 cm. The mould is inverted, top plug removed, plunger inserted and further compacted by 15 blows of the rammer. In the case of medium grained soils 25 blows each of the rammer are given instead of 15 blows. The specimen is demoulded as before.

After demoulding, the specimens are coated with paraffin wax or placed in sealed curing tins to keep the moisture content constant and they are stored in a room for a total period of 7 days at a temperature of 27±2°C. After curing, the specimens are weighed and then wax coating is removed from the flat faces before testing. (Curing of specimens may be done even without coating with wax, but by keeping them in a humid chamber). The specimens curred for 7 days are then weighed and tested in compression machine at a rate of 1.25 mm/minute and the maximum load is recorded. After compression test the moisture content of each specimen is determined from the fragments. The dry density of the specimens may be calculated for a check.

The test is repeated with remaining percentages of cement.

Mix Design

By the British method of soil-cement mix design, the average values of compressive strength are reported for each batch of five or ten specimens tested with each cement content. In order to determine the cement content of the mix which 'would give' an average compressive strength of 17.5 kg/cm² a graph is plotted with cement content versus average compressive strength. The cement content corresponding to this strength value is obtained from the graph. See Figure 9.1.

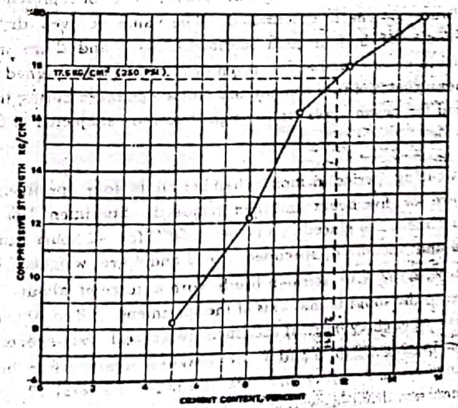

Figure 9.1 Soil-Cement Mix Design

Discussion

A compressive strength of soil-cement mixes equal to 17.5 kg/cm² at 7 days curing is considered to be satisfactory for light and medium traffic and normal climatic condition, at the U.K. For severe climatic conditions or when the traffic is too heavy, a higher compressive strength of the mix in the order of 28 to 35 kg/cm²/cm² is considered desirable. However for base course of highway pavements normaly seven days compressive strength of 17.5 kg/cm² may be adequate. A soil-cement mix which has sufficient strength is considered to be capable of withstanding the weathering effects also satisfactorily. No such strength criteria have been fixed in this country for soil-cement mix design.

WETTING AND DRYING TEST

Apparatus

(i) Cylindrical mould of 1000 ml capacity, metal rammer of weight 2.6 kg, straight edge and sample extruder used in I.S. soil compaction test.

(ii) Thermostatically controlled oven, moisture chamber to maintain 25 to 30°C and 100 percent relative humidity and water bath to submerge compacted specimens.

(iii) Wire scratch brush made of 50 × 1.6 mm flat 0.45 mm thick wire bristles assembled in groups of 10 bristles and mounted to form 5 longitudinal and 10 transverse rows of bristles on a 190×65 mm hard wood block.

(iv) Miscellaneous equipment like moisture containers, measuring device, sieve, mixing tools, scarifier, etc.

Procedure

Soil sample is prepared to provide four compacted specimens. The soil is mixed with the specified proportions of cement and potable water and the soil is compacted in three equal layers in the mould with 25 blows of the rammer on each layer. (Trial quantities of cement of varying proportions by weight may be chosen and the appropriate quantities of water to make upto OMC may be taken to prepare the mixes). The compacted specimen are trimmed and weighed. About 100 g of representative mixture is taken from the batch and the moisture content is determined. The value of oven dry density of the compacted specimen are then calculated. Of the four such specimens, No. 1 and 2 are measured to determine the volume and they are cured in moist chamber for seven days and again weighed and measured. These two specimens are used for measuring moisture content and volume changes during the wet-dry tests. Specimen Nos. 3 and 4 are also cured for seven days and weighed and are subjected to brushing and loss in weight during wet-dry test.

After the seven days curing period in moist chamber all the four specimens are submerged in potable water at room temperature for five hours and then removed. Specimen Nos. 1 and 2 are weighed and measured and the four specimens are placed in an oven at 70°C for 42 hours and then removed. Specimen No. 1 and 2 are weighed and measured, specimen No. 3 and 4 are weighed and then given two firm strokes each on all areas with the wire scratch brush with a force of about 1.4 kg, by holding the brush with the long axis parallel to the longitudinal axis of the specimens. 18 to 20 vertical brush strokes may be required to cover the sides of each cylindrical specimen twice and four strokes may be required to cover each circular end. The specimen No. 3 and 4 are weighed again after brushing. This procedure constitute one cycle of wetting and drying and are continued for total 12 cycles of wetting and drying. After 12 cycles the specimens are dried in oven at 110°C upto constant weight and then weighed.

Calculations

The volume changes of specimen No. 1 and 2 during the cycles are calculated as percentages of original volume.

The moisture contents of specimen No. 1 and 2 are calculated during the cycles as percentages of the original oven-dry weight of specimen.

The oven dry weights of specimen No. 3 and 4 are corrected for water that has reacted with the cement and soil during the test and is obtained from the relation :

$$\text{Corrected oven-dry weight} = \frac{100\ W_d}{(w - 100)}$$

where W_d = oven-dry weight after drying at 110°C, and

w = percentage of water retained in the specimens.

Soil-cement loss of specimen No. 3 and 4 is calculated as percentage of the original oven-dry weight of the specimen as follows :

$$\text{Soil cement loss, percent } 100 = \frac{A}{B}$$

where A = Original calculated oven-dry weight minus final corrected oven-dry weight

B = Original calculated oven-dry weight.

FREEZING AND THAWING TEST

Apparatus

The same apparatus used in 9.2 Wetting and Drying test are used; but instead of the water bath a freezing cabinet capable of maintaining −23°C is required and in addition absorptive pads made of 5 mm thick felt pads are needed.

Procedure

Four soil-cement specimens are prepared as in Wetting and Drying test. After 7 days curing period in moist chamber, the specimens are placed in carriers over water-saturated felt pads and the assembly is placed in freezing cabinet maintained at temperature not warmer than −23°C for 24 hours. The specimens are taken out and No. 1 and 2 specimens are weighed and measured. The four specimens are then placed in a moist chamber maintained at 25 to 30°C and 100 percent relative humidity for 23 hours. They are removed and No. 1 and 2 specimens are weighed and measured. No. 3 and 4 specimens are weighed and given two firm strokes each on all areas by the wire-scratch brush (as in wetting and drying test). The brushed specimens are weighed and turned end for end before they are placed again on water saturated pads. The above procedure consisting of one cycle of freezing and thawing is continued for total 12 cycles.

After 12 cycles of test all the specimens are dried to constant weight at 110°C and weighed to determine the oven-dry weight of the specimens.

Calculations

The volume and moisture changes of No. 1 and 2 specimens and the soil-cement losses due to brushing of No. 3 and 4 specimens are calculated as for Wetting and Drying tests.

Report

The report of the durability tests consisting of the wet-dry and freeze-thaw tests should include the cement content, OMC, maximum dry density, maximum changes in moisture and volume and total soil-cement loss in 12 cycles. These values are to be reported for each of the cement content used.

Mix Design

The PCA design method of soil-cement mixes is based on the results of wet-dry and freeze-thaw tests. The values of maximum volume change, cement content and soil-cement loss during brushing during the 12 cycles of wet-dry and freeze-thaw tests are noted for each cement content of the soil-cement mix. The cement content values are plotted against the above values for each mix.

The PCA criteria for maximum allowable brushing losses during the 12 wet-dry and freeze-thaw cycles for soils belonging to various classification groups (By Revised PRA or HRB soil classification system) are given below :

Soil groups	Maximum brushing loss
A_1, A_3, A_{2-4} and A_{2-5}	14 percent
A_{2-6}, A_{2-7}, A_4 and A_5	10 percent
A_6 and A_7	7 percent

Further as per PCA mix design criteria the maximum allowable volume change during the wet-dry and freeze-thaw cycles should not exceed 2 percent and maximum water content should not exceed the saturation moisture content of as-moulded specimens.

Applications of Soil-cement mix Design Test

The soil-cement which is to be used for base course or sub-base course of pavements should be designed in such a way that it possesses sufficient strength and durability. Up to this time there is no rational approach for the design of thickness requirements of the soil-cement base course. This is mainly because soil-cement behaves partially as a semi-rigid material. The soil-cement is generally provided equivalent to granular base course thickness. However some agencies design the thickness of granular base course of pavement for the prevailing conditions and 75 to 80 percent of this thickness is adopted for the soil cement base course. The design criteria of soil-cement mix adopted may also be specified in such cases so that the behaviour of the material is known.

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PROBLEMS

1. What is soil cement ?
2. What are the uses of soil-cement stabilization ?
3. What are the requirements of designing a soil-cement mix ?
4. What are the commonmethods of designing soil-cement mix ?
5. What are the effects of cement content on the properties of soil-cement ?
6. How is the cement requirement found based on compression strength test ?
7. What is the specified rate of loading in soil-cement specimens ?
8. Explain the wetting and drying tests on soil-cement specimen ?
9. How are freezing and thawing tests carried out on soil-cement ?
10. Specify the PCA Mix design criteria for soil-cement ?
11. What are the uses of soil-cement mix design ?

OBSERVATION SHEET

SOIL CEMENT MIX DESIGN TEST

I Compaction Test on Soil Cement Mixes

(i) Volume of the compaction mould = V cc =

(ii) Soil type =

Soil-Cement	Weight of mould, g	Weight of mould plus compacted soil.cement g	Weight of compacted soil-cement, g	Weight of moisture container, g	Weight of container plus wet soil-cement, g	Weight of container plus dry soil-cement, g	Moisture content of the soil-cement percentage	Dry density of soil, g/cm³	Maximum dry density and OMC (from graph)
Soil + x_1 % Cement									
Soil + x_2 % Cement									
Soil + x_3 % Cement									
Soil + x_4 % Cement									
Soil + x_5 % Cement									

OBSERVATION SHEET

II Compressive Strength of Soil-Cement mixes.

Type of soil :

Period of curing :

Size of mould :

Proving ring calibration factor :

Type of curing :

Cement content, %	Max. proving ring dial divisions — Test Sample Number 1	2	3	4	5	Mean	Compressive Strength, kg/cm²
x_1 =							
x_2 =							
x_3 =							
x_4 =							
x_5 =							

Cement content corresponding to 17.5 kg/cm² compressive strength from the graph = %

Remarks :

OBSERVATION SHEET

Wet-Dry and Freeze-Thaw Tests

Maximum dry density = g/cm³ ; OMC = %

Saturation Moisture Content = % ; Soil Classification group =

Cement Content, %	Maximum volume change, %		Maximum water content %		Maximum soil-cement loss due to brushing, %	
	W-D test	F-T test	W-D test	F-T test	W-D test	F-T test
x_1						
x_2						
x_3						
x_4						
x_5						

Cement requirement based on brushing loss =

Cement requirement based on volume change =

Design cement content =

Maximum water content at design cement content =

Part II

Tests on Road Aggregates

Introduction

Aggregate forms the major part of the pavement structure and it is the prime material used in pavement construction. Aggregates have to primarily bear load stresses occuring on the roads and runways and have to resist wear due to abrasive action of traffic. Aggregates are used in construction of pavements using cement concete, bituminous construction methods and in water bound macadam. Aggregates often serve as granular base course underlying the superior pavements. Thus the properties of the aggregates are of considerable significance to the highway engineers.

Aggregates which are used in the surface course have to withstand the high magnitude of load stresses and wear and tear due to abrasion. Such aggregates should possess sufficiently high strength or resistance to crushing. These aggregates further need to be hard enough to resist the wear due to abrasive action of traffic.

The aggregates in the pavements are also subjected to impact due to the moving wheel loads. Severe impact like hammering is quite common when heavily loaded steel tyred vehicles move on water bound macadam roads where stones protrude out specially in the monsoon. Jumping of the steel tyred wheels from one stone to another at different levels causes severe impact on the stones. The resistance to impact or toughness is hence another desirable property of aggregates. The stones should retain the strength and hardness and should not disintegrate under adverse weather conditions including alternate wet-dry and freeze-thaw cycles, or in other words the stones should have enough durability.

The specific gravity of stones is considered to be a measure for finding the suitability and strength characteristics of aggregates. Higher the specific gravity, better is the road stone. The presence of air voids or pores in stones also may indicate the suitability and strength characteristics of the stones. More the voids, the lesser is the specific gravity and the strength of such stones will also be lower. In bituminous road construction to some extent the presence of pores in aggregates is considered to be good for proper binding.

The size of the aggregate is qualified by the size of square sieve opening through which the same may pass, and not by the shape. All aggregates which happen to fall in a particular size range may not have the same strength and durability when compared with cubical, angular or rounded particles of the same stone. Too flaky and elongated aggregates should be avoided as far as possible as they can be crushed under the roller and traffic loads. Rounded aggregate may be preferred in cement concrete mix due to better workability for the same proportion of cement paste and same water-cement ratio, whereas rounded particles are not preferred in granular base course and water bound macadam construction as the stability due to interlocking is lesser in these aggregates; in such construction angular particles are preferred.

Heavy moving loads on the surface of flexible pavements may cause some temporary deformation of the pavement layers resulting in possible relative movement and mutual rubbing of aggregate particles. This can cause wear on the points of contacts of the aggregate in the granular base course of flexible pavements. The mutual rubbing action of the aggregates is termed as attrition and the resistance to wear due to attrition was earlier assessed by an attrition test. However this test was dropped later-on and an aggregate abrasion test alone is now considered necessary to assess the hardness of coarse aggregates.

Affinity of aggregates to bituminous binders is an important property only for the bituminous pavements. If the bituminous binder does not have affinity to the aggregates, then *stripping* is likely to occur.

The desirable properties of the aggregates may be summarised as follows :

(i) Resistance to crushing or strength
(ii) Resistance to abrasion or hardness
(iii) Resistance to impact or toughness
(iv) Good shape factors to avoid too flaky and elongated particles of coarse aggregates
(v) Resistance to weathering or soundness
(vi) Good adhesion with bituminous materials in presence of water or less stripping.

The required properties of aggregates depend on type of pavement construction, trafic and climatic conditions. All the above mentioned properties need not necessarily be possessed by aggregates for a particular construction. Engineer-in-charge will use his discretion in deciding the relative importance of these properties.

Tests which are generally carried out for judging the suitability of stone aggregates are listed below :

Strength

Aggregate crushing test

Hardness

(a) Los Angeles abrasion test (b) Deval abrasion test (c) Dorry abrasion test

Toughness

Aggregate impact test

Durability

Soundness test—accelerated durability tests

Shape factors

Shape tests

Specific gravity and Porosity

Specific gravity test and Water absorption test

Adhesion of bitumen

Striping value of aggregate

Standard tests commonly performed have been discussed in the subsequent portion.

PROBLEMS

1. What are the desirable properties of road aggregates in general ?
2. What are the different tests for judging the suitability of aggregates for use in
   (a) cement concrete pavements
   (b) W.B.M. construction
   (c) bituminous pavements
   (d) granular base course of a flexible pavement.

10

Aggregate Crushing Value Test

INTRODUCTION

The principal mechanical properties required in road stones are (i) satisfactory resistance to crushing under the roller during construction and (ii) adequate resistance to surface abrasion under traffic. Also surface stresses under rigid tyre rims of heavily loaded animal drawn vehicles are high enough to consider the crushing strength of road aggregates as an essential requirement in India.

Crushing strength of road stones may be determined either on aggregates or on cylinderical specimens cut out of rocks. These two tests are quite different in not only the approach but also in the expression of the results.

Aggregates used in road construction, should be strong enough to resist crushing under traffic wheel loads. If the aggregates are weak, the stability of the pavement structure is likely to be adversely affected. The strength of coarse aggregates is assessed by aggregates crushing test. The aggregate crushing value provides a relative measure of resistance to crushing under a gradually applied compressive load. To achieve a high quality of pavement, aggregate possessing low aggregate crushing value should be preferred.

Apparatus

The apparatus for the standard aggregate crusing test consists of the following :

(i) Steel cylinder with open ends, and internal diameter 15.2 cm, square base plate, plunger having a piston of diameter 15 cm, with a hole provided across the stem of the plunger so that a rod could be inserted for lifting or placing the plunger in the cylinder.

(ii) Cylindrical measure having internal diameter of 11.5 cm and height 18 cm.

(iii) Steel tamping rod with one rounded end, having a diameter of 1.6 cm and length 45 to 60 cm.

(iv) Balance of capacity 3 kg with accuracy upto 1 g.

(v) Compressions testing machine capable of applying load of 40 tonnes, at a uniform rate of loading of 4 tonnes per minute.

Figure 10.1 shows the aggregate crushing test apparatus.

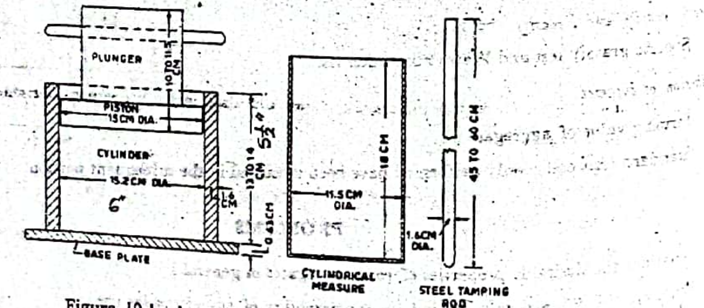

Figure 10.1 Aggregate Crushing Test Apparatus

Procedure

The aggregate passing 12.5 mm IS sieve and retained on 10 mm IS sieve is selected for standard test. The aggregate should be in surface-dry condition before testing. The aggregate may be dried by heating at a temperature 100°C to 110°C for a period of 4 hours and is tested after being cooled to room temperature.

The cylindrical measure is filled by the test sample of aggregate in three layers of approximately equal depth, each layer being tamped 25 times by the rounded end of the tamping rod. After the third layer is tamped, the aggregates at the top of the cylindrical measure is levelled off by using the tamping rod as a straight edge. About 6.5 kg of aggregate is required for preparing two test samples. The test sample thus taken is then weighed. The same weight of the sample is taken in the repeat test.

The cylinder of the test apparatus is placed in position on the base plate; one third of the test sample is placed in this cylinder and tamped 25 times by the tamping rod. Similarly, the other two parts of the test specimen are added, each layer being subjected to 25 blows. The total depth of the material in the cylinder after tamping shall however be 10 cm. The surface of the aggregates is levelled and the plunger inserted so that it rests on this surface in level position. The cylinder with the test sample and the plunger in position is placed on compression testing machine. Load is then applied through the plunger at a uniform rate of 4 tonnes per minute until the total load is 40 tonnes, and then the load is released. Aggregates including the crushed portion are removed from the cylinder and sieved on a 2.36 mm IS sieve. The material which passes this sieve is collected.

The above crushing test is repeated on second sample of the same weight in accordance with above test procedure. Thus two tests are made for the same specimen for taking an average value.

Calculation

Total weight of dry sample taken = W_1 g. Weight of the portion of crushed material passing 2.36 mm IS sieve = W_2 g.

The aggregate crushing value is defined as a ratio of the weight of fines passing the specified IS sieve to the total weight of the sample expressed as a percentage. The value is usually recorded up to the first decimal place.

$$\text{Aggregate crushing value} = \frac{100\, W_2}{W_1}$$

Results

The mean of the crushing value obtained in the two tests is reported as the aggregate crushing value.

Determination of Ten Percent Fines Value

The 'ten percent fines' value is a measure of resistance of the aggregates to the crushing. The apparatus and materials used are the same as for the standard aggregate crushing test. The test sample in the cylinder with the plunger in position is placed in the compression testing machine. The load is applied at a uniform rate so as to cause a total penetration of the plunger of about 20 mm for normal crushed aggregates in 10 minutes. But for rounded or partially rounded aggregates, the load required to cause a total penetration of 15 mm is applied where as for honeycombed aggregate like expanded shales or slags that for a total penetration of 24 mm is applied in 10 minutes. After the maximum specified penetration is reached, the load is released and the aggregates from the cylinder is sieved on a 2.36 mm IS sieve. The fines passing this sieve is weighed and is expressed as a percentage by weight of the test sample. This percentage normally falls in the range of 7.5 to 12.5; but if it does not fall in this range, the test is repeated with necessary adjustment of the load.

Two tests are carried out at the load (x tonnes) which give the percentage fines between 7.5 and 12.5 and let the mean of the percent fines be 'y' for calculating the load required for 10 percent fines.

$$\text{Load for 10 percent fine} = \frac{14x}{(y-4)}$$

Discussion

In general, large size of aggregates used in the test results in higher aggregates crushing value. The relationship between the aggregate sizes and the crushing values will however vary with the type of specimens tested. When non-standard sizes of aggregates are used for the crushing test, (i.e. aggregate larger than 12.5 mm or smaller than 10 mm) the size of the cylinder, quantity of material for preparation of specimen size of IS sieve for separating fines and the amount and rate of compaction shall be adopted as given in Table 10.1.

TABLE 10.1

Details for Aggregate Crushing Test with Non-standard Sizes of Aggregates

Aggregate size: Passing sieve size, mm	Aggregate size: Retained on sieve size, mm	Diameter of cylinder to be used, cm	Quantity of material and preparation of test sample	Loading	Size of IS sieve for separating fines
25	20	15 *(standard cylinder)	**Standard method	+Standard loading	4.75 mm
				Standard loading	3.35 mm
20	12.5	"	Standard method		
10	6.3	7.5	Metal measure 5 cm dia & 9 cm height tamping rod 8mm dia 30cm long depth of material in 7.5 cm cylinder after tamping 5 cm	Rate of loading one tonne per min. upto a total load of 10 tonnes.	1.70 mm
6.3	4.75	7.5	as above	as above	1.18 mm
4.75	3.35	7.5	as above	as above	850 microns
3.35	2.36	7.5	as above	as above	600 microns

*Standard cylinder as given in Figure 10.1.
**Standard method of preparing sample as given in procedure.
+Standard loading as given in procedure

The aggregate sample for conducting the aggregate crushing test for the first time is to be taken by volume in the specified cylindrical measure by tamping in a specified manner and the weight of this sample is determined. When the test is repeated using the same aggregate, it is sufficient tod irectly weigh and take the same weight of sample. This is because it is necessary to keep the volume and height of the test specimens in the aggregate crushing mould constant when testing any aggregate sample so that the test conditions remain unaltered. If the quantity of test sample to be taken is specified by weight, the volume and hence the height may vary depending on the variation in specific gravity and shape factors of different aggregates.

When aggregates are not available, crushing strength test may be carried out on cylindrical specimen prepared out of rock sample by drilling, sawing and grinding. The specimen may be subjected to a slowly

increasing compressive load until failure to find the crushing strength in kg/cm². However this test is seldom carried out due to difficulty in preparing specimens and not getting reproducible results. On the contrary the aggregate crushing test is simple, rapid and gives fairly consistent results.

Applications of Aggregate Crushing Test

The aggregate crushing value is an indirect measure of crushing strength of the aggregates. Low aggregate crushing value indicates strong aggregates, as the crushed fraction is low. Thus the test can be used to assess the suitability of aggregates with reference to the crushing strength for various types of pavement components. The aggregates used for the surface course of pavements should be strong enough to withstand the high stresses due to wheel loads, including the steel tyres of loaded bullock-carts. However as the stresses at the base and sub-base courses are low, aggregates with lesser crushing strength may be used at the lower layers of the pavement.

Indian Roads Congress and ISI have specified that the aggregate crushing value of the coarse aggregates used for cement concrete pavement at surface should not exceed 30 percent. For aggregates used for concrete other than for wearing surfaces, the aggregate crushing value shall not exceed 45 percent, according to the ISS. However aggregate crushing values have not been specified by the IRC for coarse aggregates to be used in bituminous pavement construction methods.

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PROBLEMS

1. How is the crushing strength test carried out on cylindrical stone specimen? Why is the test not carried out commonly?
2. Explain aggregate crushing value. How would you express it?
3. Briefly explain the aggregate crushing value test procedure.
4. What is the specified standard size of aggregates? How is the aggregate crushing value of non-standard size aggregate evaluated?
5. Aggregate crushing value of material A is 40 and that of B is 25. Which one is better and why?
6. What are the applications of aggregate crushing test?
7. What are the recommended maximum values of aggregate crushing value for the aggregates to be used in base and surface courses of cement concrete?
8. What are the uses and applications of the aggregate crushing test?

OBSERVATION SHEET

AGGREGATE CRUSHING VALUE TEST

Type of specimen (Description giving source, particle shape and surface texture)

Sample Number	Total weight of dry Sample, W_1 g	Weight of fines passing 2.36 mm IS Sieve, W_2 g	Aggregate crushing value $= \frac{W_2}{W_1} \times 100$ percent	Average aggregate crushing value = Average of col. (4)
(1)	(2)	(3)	(4)	(5)
1				
2				

Result : Aggregate crushing value = percent

Remarks :

II
Abrasion Test

INTRODUCTION

Due to the movement of traffic, the road stones used in the surfacing course are subjected to wearing action at the top. Resistance to wear or hardness is hence an essential property for road aggregates, especially when used in wearing course. Thus road stones should be hard enough to resist the abrasion due to the traffic. When fast moving traffic fitted with pneumatic tyres move on the road, the soil particles present between the wheel and road surface causes abrasion on the road stone. Steel tyres of animal drawn vehicles which rub against the stones can cause considerable abrasion of the stones on the road surface. Hence in order to test the suitability of road stones to resist the abrading action due to traffic, tests are carried out in the laboratory.

Abrasion tests on aggregates are generally carried out by any one of the following methods :

(i) Los Angeles abrasion test

(ii) Deval abrasion test

(iii) Dorry abrasion test

Of these tests, the Los Angeles abrasion test is more commonly adopted as the test values of aggregates have been correlated with performance of studies. The ISI has suggested that wherever possible, Los Angeles abrasion test should be preferred.

In addition to the above abrasion tests, another test which is carried out to test the extent to which the aggregates in the wearing surface get polished under traffic, is *'Polished Stone Value'* test. Samples of aggregates are subjected to an accelerated polishing test in a machine and a friction test is carried out on the polished specimen. The results of this test are useful only for comparative purpose and specifications are not yet available.

LOS ANGELES ABRASION TEST

Introduction

The principle of Los Angeles abrasion test is to find the percentage wear due to the relative rubbing action between the aggregates and steel balls used as abrasive charge; pounding action of these balls also exist while conducting the test. Some investigators believe this test to be more dependable as rubbing and pounding action simulate the field conditions where both abrasion and impact occur. Los Angeles abrasion test has been standardised by the ASTM, AASHO and also by the ISI. Standard specification of Los Angeles abrasion values are also available for various types of pavement constructions.

Apparatus

The apparatus consists of Los Angeles machine and sieves.

Los Angeles machine consists of a hollow steel cylinder, closed at both ends, having an inside diameter 70 cm and an inside length of 50 cm, mounted on stub shafts about which it rotates on a horizontal axis.

An opening is provided in the cylinder for the introduction of the test sample. A removable cover of the opening is provided in such a way that when closed and fixed by bolts and nuts, it is dust-tight and the interior surface is perfectly cylindrical. A removable steel shelf projecting radially 8.8 cm into the cylinder and extending to the full length of it, is mounted on the interior surface of the cylinder rigidly, parallel to the axis. The shelf is fixed at a distance of 125 cm from the opening, measured along the circumference in the direction of rotation. Refer Figure 11.1. Abrasive charge, consisting of cast iron spheres approximately 4.8 cm in diameter and 390 to 445 g in weight are used. The weight of the sphere

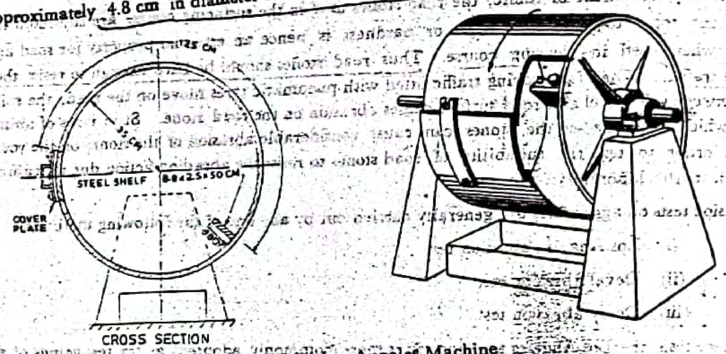

Figure 11.1 Los Angeles Machine

used as the abrasive charge and the number of spheres to be used are specified depending on the gradation of the aggregates tested. The aggregate grading have been standardised as A, B, C, D, E, F, and G for this test and the IS specifications for the grading and abrasive charge to be used are given in Table 11.1. IS sieve with 1.70 mm opening is used for separating the fines after the abrasion test.

Procedure

Clean aggregates dried in an oven at 105—110°C to constant weight, conforming to any one of the grading A, to G, as per Table 11.1, is used for the test. The grading or gradings used in the test should be nearest to the grading to be used in the construction. Aggregates weighing 5 kg for grading A, B, C or D and 10 kg for gradings E, F or G may be taken as test specimen and placed in the cylinder. The abrasive charge is also chosen in accordance with Table 11.1 depending on the grading of the aggregate and is placed in the cylinder of the machine. The cover is then fixed dust-tight. The machine is rotated at a speed of 30 to 33 revolutions per minute. The machine is rotated for 500 revolutions for gradings A, B, C and D, for gradings E, F and G, it shall be rotated for 1,000 revolutions. The machine should be balanced and driven in such a way as to maintain uniform peripheral speed.

After the desired number of revolutions, the machine is stopped and the material is discharged from the machine taking care to take out entire stone dust. Using a sieve of size larger than 1.70 mm IS sieve the material is first separated into two parts and the finer position is taken out and sieved further on 1.7 mm IS sieve. The portion of material coarser than 1.7 mm size is washed and dried in an oven at 105 to 110° to constant weight and weighed correct to one gram.

Calculations

The difference between the original and final weights of the sample is expressed as a percentage of the original weight of the sample is reported as the percentage wear.

TABLE 11.1

Specifications for Los Angeles Test

Grading	Weight in grams of each test sample in the size range, mm (Passing and retained on square holes.)										Abrasive charge	
	80—63	63—50	50—40	40—25	25—20	20–12.5	12.5–10	10–6.3	6.3–4.75	4.75–2.36	Number of spheres	Weight of charge, g
A	—	—	—	1250	1250	1250	1250	—	—	—	12	5000±25
B	—	—	—	—	—	2500	2500	—	—	—	11	4584±25
C	—	—	—	—	—	—	—	2500	2500	—	8	3330±20
D	—	—	—	—	—	—	—	—	—	5000	6	2500±15
E	2500*	2500*	5000*	—	—	—	—	—	—	—	12	5000±25
F	—	—	5000*	5000*	—	—	—	—	—	—	12	5000±25
G	—	—	—	5000*	5000*	—	—	—	—	—	12	5000±25

*Tolerence of ± 2 percent is permitted.

Let the original weight of aggregate $= W_1$ g

Weight of aggregate retained on 1.70 mm IS sieve after the test $= W_2$ g

Loss in weight due to wear $= (W_1 - W_2)$ g

Los Angeles abrasion value, % = Percentage wear $= \frac{(W_1 - W_2)}{W_1} 100$

Results

The result of the Los Angeles abrasion test is expressed as a percentage wear and the average value of two tests may be adopted as the Los Angeles abrasion value.

Discussion

It may seldom happen that the aggregates desired for a certain construction project has the same grading as any one of the specified gradings. In all the cases, standard grading or gradings nearest to the gradation of the selected aggregates may be chosen.

Different specification limits may be required for gradings E, F and G, when compared with A, B, C and D. Further investigations are necessary before any such specifications could be made.

Los Angeles abrasion test is very commonly used to evaluate the quality of aggregates for use in pavement construction, especially to decide the hardness of stones. The allowable limits of Los Angeles abrasion values have been specified by different agencies based on extensive performance studies in the field. The ISI has also suggested that this test should be preferred wherever possible. However, this test may be considered as one in which resistance to both abrasion and impact of aggregate may be obtained simultaneously, due to the presence of abrasive charge. Also the test condition is considered more representative of field conditions. The results obtained on stone aggregates are highly reproducible.

Applications of Los Angeles Abrasion Test

Los Angeles Abrasion test is very widely accepted as a suitable test to assess the hardness of aggregates used in pavement construction. Many agencies have specified the desirable limits of the test, for different methods of pavement construction. The maximum allowable Los Angeles abrasion values of aggregates specified by Indian Roads Congress for different methods of construction are given in Table 11.2.

TABLE 11.2

Maximum Allowable Los Angeles Abrasion Values of Aggregates in Different Types of Pavement Layers

Serial No.	Types of pavement layer	Los Angeles abrasion value, maximum %
1.	Water Bound Macadam (WBM), sub-base course	60
2.	(i) WBM base course with bituminous surfacing (ii) Bituminous Macadam base course (iii) Built-up spray grout base course	50
3.	(i) WBM surfacing course (ii) Bituminous Macadam binder course (iii) Bituminous penetration Macadam (iv) Built-up spray grout binder course	40
4.	(i) Bituminous carpet surface course (ii) Bituminous surface dressing, single or two coats (iii) Bituminous surface dressing, using precoated aggregates (iv) Cement concrete surface course (as per IRC)	35
5.	(i) Bituminous/Asphaltic concrete surface course (ii) Cement concrete pavement surface course (as per ISI)	30

DEVAL ABRASION TEST

Introduction

Deval abrasion test was devised to test rock fragments. Later this test has been standardised by ASTM for finding the rate of wear of stone aggregates, by crushing them to tumble one over other in a rattler in presence of abrasive charge. Deval abrasion test has also been standardised by IST as a test for abrasion of coarse aggregates. In this test also both abrasion and impact take place due to the steel balls used as abrasive charge.

Apparatus

The apparatus for the test consists of the Deval machine and standard sieve.

The deval abrasion testing machine consists of one or more (generally two) hollow cast iron cylinders closed at one end and provided with iron cover at the other end, capable of fitting tightly. The inside diameter of the cylinder is 20 cm and length is 34 cm. The cylinders are mounted on a shaft at an angle of 30 degrees with the axis of rotation. See figure 11.2. Cast iron or steel spheres of about 4.8 cm diameter and 390 to 445 g weight are used as abrasive charge. Six such spheres are used as abrasive charge, their total weight being 2500 ± 10 g.

IS sieves having 1.70 mm square holes are used for sieving the materials after the abrasion test.

Procedure

Test sample consists of dry coarse aggregates made of different percentages of the various sizes

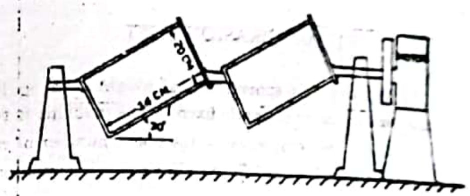

Figure 11.2 Deval's Machine

conforming to any one of the gradings given in Table 11.3. The material is washed, dried and separated to different sizes by sieving. The grading adopted for the test should be the one which most nearly represents the coarse aggregate to be used for a particular construction project. Crushed gravel conforming to the above specifications can also be used.

TABLE 11.3

Grading of Aggregates for Deval Abrasion Test

Grading	Passing IS sieve, mm	Retained on IS sieve mm	Percentage of sample
A	20	12.5	25
	25	20.0	25
	40	25.0	25
	50	40.0	25
B	20	12.5	25
	25	20.0	25
	40	25.0	50
C	20	12.5	50
	25	20.0	50
D	12.5	4.75	50
	20	12.5	50
E	10	4.75	50
	12.5	10.00	50

The weight of the sample to be taken for the test depends on its average specific gravity and is given in Table 11.4:

Table 11.4 Weight of Sample for Deval Abrasion Test

Range of specific gravity	Weight of sample, g
Over 2.8	5,500
*2.4 to 2.8	5,000
2.2 to 2.39	4,500
less than 2.2	4,000

*Most commonly found

DEVAL ABRASION TEST

The sample and the abrasive charge of 6 spheres of total weight 2,500 ± 10 grams are placed in the Deval abrasion testing machine and the cover is tightly fixed. The machine is rotated at a speed of 30 to 33 r.p.m. for 10,000 revolutions. At the completion of the above number of revolutions, the material is removed from the machine and is sieved on a 1.70 mm IS sieve. The material retained on the sieve is washed, dried and weighed to the nearest gram.

Calculation

(i) The loss in weight by abrasion is the difference between the original weight of the test sample and the weight of material retained on the 1.70 mm IS sieve after the test. The percentage of wear is the loss in weight by abrasion expressed as a percentage of the original weight of the sample.

Let the original weight of the sample be $= W_1$ g

Weight of material retained on 1.70 mm IS sieve after the abrasion test $= W_2$ g

Therefore percentage wear or Deval abrasion value, % $= \left(\frac{W_1 - W_2}{W_1}\right) 100$

(ii) In the case of crushed gravel (i.e., fragment of gravel having atleast one fractured face) the percentage by weight of crushed fragments should be determined and the permissible percentage wear is calculated as given below.

$$W = \frac{AL + (100 - A) L'}{100}$$

Where W = permissible percentage of wear

A = percentage of uncrushed fragments

L = maximum percentage of wear permitted by the specifications for aggregates consisting entirely of crushed fragments.

(100 – A) = percentage of crushed fragments.

and L′ = maximum percentage of wear permitted by the specifications of gravel consisting entirely of crushed fragments.

Results

Duplicate test may be carried out simultaneously by placing similar specimens in the second cylinder and the average values of the two tests may be calculated. The report includes (a) percentage of wear. (b) percentage of crushed fragments in the test sample and (c) weight and grading of the test sample.

Discussion

When coarse aggregates furnished for the work contains as much as 25 percent of material finer than 12.5 mm but is of such size that either grading A, B or C would be used for the abrasion test, a second abrasion test should be carried out, using grading D, if in the opinion of the engineer, the particles lesser than 12.5 mm size are not atleast equal in hardness to those particles greater than 12.5 mm size.

The British *attrition* test using Deval's machine is similar to the rattler type of test explained in this experiment with an exception that no abrasive charge is used. Deval abrasion test is in fact a modified Deval's attrition test, using abrasive charge. The attrition test which was formely standardised by BSI has been omitted, later on as of doubtful value.

Applications of Deval Abrasion Test

It has been recommended by the ISI that where ever possible the Los Angeles abrasion test should be preferred to the Deval abrasion test. The desirable limits of percentage wear by the Deval abrasion test

have not been specified by agencies, as this is not a common test. Thus the test has limited uses and applications.

DETERMINATION OF AGGREGATE ABRASION VALUE BY BRITISH TEST

Introduction

This test for the determination of aggregate abrasion value has been standardised by the British Standards Institution. Formerly the Dorry abrasion test was devised for testing the resistance to abrasion of cylindrical stone specimens on a rotating metal disc in presence of sand used as abrading agent. Now the test has been modified so as to find the abrasion value of aggregates.

Apparatus

The apparatus consists of the Dorry abrasion machine and accessories and a set of B.S. test sieves.
The abrasion machine has a flat circular cast iron or steel disc, not less than 60 cm in diameter which can
be rotated in a horizontal plane at a speed of 28 to 30 r.p.m. Two trays made from 3 mm mild steel plate,
of internal dimensions 95×57×8 mm are used for holding the smaller trays with samples. These two
trays are to be held with their centre points 26 cm from the centre of disc, diametrically opposite to
each other and with their long side placed in the direction of rotation of the disc. A weight of 2 kg is
used to press the test sample down on the dics. There is a device to feed standard abrasion sand continu-
ously on the disc in front of each test sample at a rate of 680 to 900 g per minute. Two smaller trays made
of mild steel of internal dimensions 92×54×8 mm are used for keeping the test sample. These smaller
trays can just fit inside the large trays meant for holding the test specimens.

B.S. test sieves of sieve openings 12.5, 8.3, 0.85, 0.6, 0.42, 0.3 and 0.15 mm are also necessary to sieve the aggregate sample, the abrasive sand and fine sand. Other apparatus are hot plate, oven, balance, etc.

Procedure

The test sample consists of clean aggregates, passing 12.5 and retained on 8.3 mm sieve, free from
flaky particicles, 33 cm³ of such dry aggregates is placed in one of the smaller trays to form a single layer
projecting 5 to 6 mm above the upper edge of the tray. The interstices between the aggregates is filled up
to the level of the top of smaller tray by fine sand passing 0.15 mm sive. The tray with the aggregate
and fine sand is heated to temperature not less than 80°C.

One of the larger trays is placed on a hot plate and is filled with a molten setting compound. The
setting compound may consist of a mixture of pitch and plaster of Paris of equal proportions. The
compound is allowed to cool till it is viscous enough to enable the tray be inverted and pressed down on the
hot aggregate in the smaller tray. The two trays are kept pressed together and cooled till the single layer of
prjecting aggregate is firmly held by the setting compound. The test sample of aggregate should have
some what flat upper surface, approximately level with the top edge of the tray. The sand and surplus
setting compound are removed and the tray with the aggregate set-in is weighed.

The sample trays are placed on the abrasion machine and loaded so that the total load including the
specimen with tray is 2 kg. Standard Buzzard Silica sand, at least 75 percent passing 0.6 mm sive and
all passing 0.85 and retained on 0.3 mm sieve is used as abrasive sand. This abrasive sand is fed
continuously on the disc of the machine which is rotated at a speed of 28–30 r.p.m. Two samples of test
specimens are tested simultaneously.

After 500 revolutions, the test samples are removed and weighed.

Calculation

The aggregate abrasion value is expressed as the percentage loss in weight due to abrasion. This is calculated from the formula :

$$\text{Percentage loss in weight} = \frac{100\,(B-C)}{A}$$

where
A = weight of oven-dry sample aggregate, g
B = weight of tray with aggregate and setting compound before abrasion, g
C = weight of tray with aggregate and setting compound after the abrasion value.

The mean of the two test results is reported as the aggregate abrasion value.

Discussion

The aggregate abrasion test given above is rapid and easy to perform, except for the preparation of test specimen, which is difficult and time consuming. This test is however carried out commonly in U.K. and a few other countries. The Dorry abrasion testing machine which was previously used to test cylindrical stone specimens can be used for conducting this aggregate abrasion test, with slight modifications for holding the specimens. In many countries, abrasion of road aggregates is assessed by the Los Angeles abrasion test as the results of this test have been extensively correlated with performance studies.

Application of British (Dorry) Aggregate Abrasion Test

The presence of sand on the pavement is considered to act as abrading material between the traffic wheels and the aggregates on the surface, causing additional wear. Hence the Dorry abrasion test results should be useful in assessing the suitability of aggregates for use in the pavement surface courses.

The aggregate abrasion value by this test is found to vary from below 1 for some Flints to over 15 for aggregates which may normally be regarded as too soft for use in wearing course of pavements. The aggregate abrasion value of Granites are found to very between 3 and 9 where as those of Basalts vary between 7 and 25 and Lime stones between 17 and 33. Aggregates with abrasion values below 5 may thus be considered quite hard varieties.

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PROBLEMS

1. Why is Los Angeles abrasion test considered superior to other tests to find the hardness of aggregates ?
2. How is Los Angeles abrasion value expressed ?
3. The abrasion value found from Los Angeles test for aggregates A and B are 35 and 15 respectively. Which aggregate is harder ? Why ? For what types of constructions are these suitable ?
4. Briefly explain the Los Angeles abrasion test procedure.
5. What are the desirable limits of Los Angeles Abrasion values specified for different types of pavement surfacings ?
6. Explain when the test carried out in Deval's machine is called an abrasion or attrition test.
7. Give the steps for Deval abrasion test.
8. How is the abrasion value expressed after testing the aggregates in Deval's machine.
9. Explain briefly how the British (Dorry) aggregate abrasion value is found.
10. Discuss the significance of sand used in the Dorry aggregate abrasion test ?
11. How is British (Dorry) aggregate abrasion value epxressed ? Two materials have abrasion values 5 and 10 respectively. Which one is harder and why ?

LOS ANGELES ABRASION TEST

(i) Type of aggregate =
(ii) Grading =
(iii) Number of spheres used =
(iv) Weight of charge =
(v) Number of revolutions =

	Test Number 1	Test Number 2	Average
1. Weight of specimen = W_1 g			
2. Weight of specimen after abrasion test, coarser than 1.70 mm IS sieve = W_2 g			
3. Percentage wear = $\frac{(W_1 - W_2)}{W_1}$ 100			

Quality of aggregate :

Remarks :

DEVAL ABRASION TEST

(i) Type of aggregate =
(ii) Percent crushed ragments =
(iii) Grading of sample =
(iv) Specific gravity of the sample =

	Test Number 1	Test Number 2	Mean value
Original weight of the sample = W_1 g			
Weight of material retained on 1.7 mm IS sieve after abrasion test = W_2 g			
Percentage wear = $\frac{(W_1 - W_2)}{W_1}$ 100			

Remarks

BRITISH (DORRY) ABRASION TEST ON AGGREGATES

Type of aggregate specimen =

	Trial Number 1	Trial Number 2	Trial Number 3
Original weight of aggregate = W_1 g			
Weight of worn aggregate after abrasion test = W_2 g			
Loss in weight due to abrasion = $(W_1 - W_2)$ g			
Abrasion value of aggregate = percent loss in weight = $\frac{(W_1 - W_2)}{W_1}$ 100 percent			

Abrasion value =

Remarks

Aggregate Impact Test

INTRODUCTION

Toughness is the property of a material to resist impact. Due to traffic loads, the road stones are subjected to the pounding action or impact and there is possibility of stones breaking into smaller pieces. The road stones should therefore be tough enough to resist fracture under impact. A test designed to evaluate the toughness of stones i.e., the resistance of the stones to fracture under repeated impacts may be called an impact test for road stones.

Impact test may either be carried out on cylindrial stone specimens as in *Page Impact* test or on stone aggregates as in Aggregate Impact test. The Page Impact test is not carried out now-a-days and has also been omitted from the revised British Standards for testing mineral aggregates. The Aggregate Impact test has been standardised by the British Standards Institution and the Indian Standards Institution.

The aggregate impact value indicates a relative measure of the resistance of an aggregate to a sudden shock or an impact, which in some aggregates differs from its resistance to a slow compressive load. The method of test covers the procedure for determining the aggregate impact value of coarse aggregates.

Apparatus

The apparatus consists of an impact testing machine, a cylindrical measure, tamping rod, IS sieves, balance and oven.

(a) Impact testing machine : The machine consists of a metal base with a plane lower surface supported well on a firm floor, without rocking. A detachable cylindrical steel cup of internal diameter 10.2 cm and depth 5 cm is rigidly fastened centrally to the base plate. A metal hammer of weight between 13.5 and 14.0 kg having the lower end cylindrical in shape, 10 cm in diameter and 5 cm long, with 2 mm chamfer at the lower edge is capable of sliding freely between vertical guides, and fall concentric over the cup. There is an arrangement for raising the hammer and allowing it to fall freely between vertical guides from a height of 38 cm on the test sample in the cup, the height of fall being adjustable upto 0.5 cm. A key is provided for supporting the hammer while fastening or removing the cup. Refer Figure 12.1.

(b) Measure : A cylindrical metal measure having internal diameter 7.5 cm and depth 5 cm for measuring aggregates.

(c) Tamping rod : A straight metal tamping rod of circular cross section, 1 cm in diameter and 23 cm long, rounded at one end.

(d) Sieve : IS sieve of sizes 12.5 mm, 10 mm and 2.36 mm for sieving the aggregates.

(e) Balance : A balance of capacity not less than 500 g to weigh accurate upto 0.1 g.

(f) Oven : A thermostatically controlled drying oven capable of maintaining constant temperature between 100°C and 110°C.

Procedure

The test sample consists of aggregates passing 12.5 mm sieve and retained on 10 mm sieve and dried in

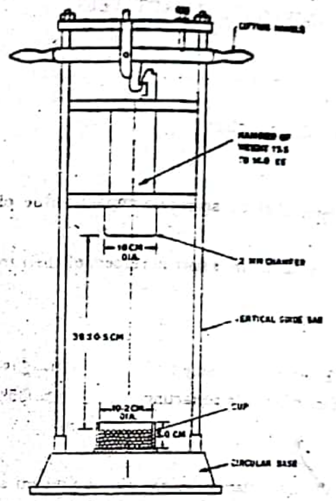

Figure 12.1 Aggregate Impact Testing Machine

an oven for four hours at a temperature 100°C to 110°C and cooled. Test aggregates are filled upto about one-third full in the cylindrical measure and tamped 25 times with rounded end of the tamping rod. Further quantity of aggregates is then added upto about two-third full in the cylinder and 25 strokes of the tamping rod are given. The measure is now filled with the aggregates to over flow, tamped 25 times. The surplus aggregates are struck off using the tamping rod as straight edge. The net weight of the aggregates in the measure is determined to the nearest gram and this weight of the aggregates is used for carrying out duplicate test on the same material. The impact machine is placed with its bottom plate flat on the floor so that the hammer guide columns are vertical. The cup is fixed firmly in position on the base of the machine and the whole of the test sample from the cylindrical measure is transferred to the cup and compacted by tamping with 25 strokes.

The hammer is raised until its lower face is 38 cm above the upper surface of the aggregates in the cup, and allowed to fall freely on the aggregates. The test sample is subjected to a total of 15 such blows, each being delivered at an interval of not less than one second. The crushed aggregate is then removed from the cup and the whole of it sieved on the 2.36 mm sieve until no further significant amount passes. The fraction passing the sieve is weighed accurate to 0.1 g. The fraction retained on the sieve is also weighed and if the total weight of the fractions passing and retained on the sieve is added it should not be less than the original weight of the specimen by more than one gram; if the total weight is less than the original by over one gram the result should be discarded and a fresh test made.

The above test is repeated on fresh aggregate sample.

Calculation

The aggregate impact value is expressed as the percentage of the fines formed in terms of the total weight of the sample.

Let the original weight of the oven dry sample be W_1 g and the weight of fraction passing 2.36 mm IS sieve be W_2 g, Aggregate impact value $= \frac{100 W_2}{W_1}$ percent.

This is recorded correct to the first decimal place.

Results

The mean of the two results is reported as the aggregate impact value of the specimen to the nearest whole number.

Aggregate impact value is used to classify the stones in respect of their toughness property as indicated below :

Aggregate impact values

< 10%	Exceptionally strong ;	10—20%	Strong.
10 — 30%	Satisfactory for road surfacing ;	> 35%	Weak for road surfacing

Discussion

Chief advantage of aggregate impact test is that the test equipment and the test procedure are quite simple and it determines the resistance to impact of stones simulating field condition. The test can be performed in a short time even at construction site or at stone quarry, as the apparatus is simple and portable.

Well shaped cubical stones provide higher resistance to impact when compared with flaky and elongated stones.

It is essential that the first specimen to be tested from each sample of aggregate is equal in volume; this is ensured by taking the specimen in the measuring cylinder in the specified manner by tamping in three layers. If all the test specimens to be tested in the aggregate impact testing mould are of equal volume, the height of these specimens will also be equal and hence the height of fall of the impact rammer on the specimens will be equal. On the other hand, if equal weight of different aggregate samples are taken, their volume and height may vary depending upon the specific gravity of the aggregates and their shape factors.

There is no definite reason why the specified rate of application of the blows of the impact rammer should be maintained.

Applications of Aggregate Impact Value

The aggregate impact test is considered to be an important test to assess the suitability of aggregates as regards the toughness for use in pavement construction. It has been found that for majority of aggregates, the aggregate crushing and aggregate impact values are numerically similar within close limits. But in the case of fine grained highly siliceous aggregate which are less resistant to impact than to crushing, the aggregate impact values are higher (on the average, by about 5) than the aggregate crushing values.

Various agencies have specified the maximum permissible aggregate impact values for the different types of pavements, those recommended by the Indian Roads congress are given in Table 12.1.

For deciding the suitability of soft aggregates in base course construction, this test has been commonly used. A modified impact test is also often carried out in the case of soft aggregates to find the wet impact value after soaking the test samples. The recommendations given in Table 12.2 based on work reported by different agencies have been made to assess the suitability of soft aggregates for road construction

TABLE 12.1

Maximum Allowable Impact Values of Aggregate in different Types of Pavement Material/Layers

Serial No.	Types of payment material/layer	Aggregate impact value, maximum, %
1	Water bound macadam (WBM), sub-base course	50
2	Cement concrete, base course (as per ISI)	45
3	(i) WBM base course with bitumen surfacing (ii) Built-up spray grout, base course	40
4	Bituminous macadam, base course	35
5	(i) WBM, surfacing course (ii) Built-up spray grout, surfacing course (iii) Bituminous penetration macadam (iv) Bituminous macadam, binder course (v) Bituminous surface dressing (vi) Bituminous carpet (vii) Bituminous/Asphaltic concrete (viii) Cement concrete, surface course	30

TABLE 12.2

Condition of sample	Maximum aggregate impact value, percent	
	Sub-base and base	Surface course
Dry	50	32
Wet	60	39

REFERENCES

1. Bituminous Materials in Road Construction, *D.S.I.R.* H.M.S.O., London.
2. Road Aggregates, their uses and Testing, *B.H. Knight*, and *R.G. Knight*, Edward Arnold Co., London.
3. Indian standard Methods of Test for Aggregate for Concrete, IS : 2386 part IV *Indian Standards Institution.*
4. Indian Standard Specification for Coarse and Fine Aggregate from Natural Sources, IS : 383, *Indian Standards Institution.*
5. Tentative Specification [For Various Types of Construction Methods], *Indian Roads Congress.*
6. Standard Specifications and code of Practice for Construction of Concrete Roads, IRC : 15-1970, *Indian Roads Congress.*
7. Methods for sampling and Testing of Aggregates, Sands and Fillers, BS : 812, *British Standard Institution.*
8. Report of the Seminar on Low Cost Roads and Soil Stabilization, *E.C.A.F.E.*, New Delhi, 1958.

PROBLEMS

1. What are the advantages of Aggregate Impact test over Page Impact test ?
2. Briefly mention the procedure of aggregate impact test ?
3. How is aggregate impact value expressed ?
4. What are the desirable limits of aggregate impact value specified for different types of pavement surfaces ?
5. Aggregate impact value material A is 20 and that of B is 45. Which one is better for surface course ? Why ?
6. What do you understand by dry and wet impact values ?

OBSERVATION SHEET

AGGREGATE IMPACT TEST

Serial No.	Details	Trial number 1	Trial number 2	Average
1	Total weight of aggregate sample filling the cylindrical measure = W_1 g			
2	Weight of aggregate passing 2.36 mm sieve after the test = W_2 g			
3	Weight of aggregate retained on 2.36 mm sieve after the test = W_3 g			
4	$(W_1 - W_2 + W_3)$ g			
5	Aggregate Impact value = percent fines $= 100 \frac{W_2}{W_1}$ percent			

Quality of Aggregate :

Remarks :

13

Soundness Test

INTRODUCTION

This test is intended to study the resistance of aggregates to weathering action. In the absence of adequate information from performance studies, a laboratory test simulating accelerated weathering condition is carried out to judge the durability or the soundness of the aggregates.

In order to quicken the effect of weathering due to alternate wet-dry and or freeze-thaw cycles in the laboratory, the resistance to disintegration of aggregate is determined by soaking the aggregate specimen in saturated solutions of sodium sulphate or magnesium sulphate.

Apparatus

The apparatus required for the test are containers for aggregates, sieves, balance, device for temperature regulation and a drying oven. The containers may be made with suitable perforations or with wire mesh to permit free access and drainage of the solution from the sample. IS sieves having square openings and of sizes 4.75, 8.0, 10.0, 12.5, 16.0, 20.0, 25.0, 31.0, 40.0, 50.0, 63.0 and 80.0 mm are necessary for testing coarse aggregates. A balance of capacity 5 kg to weight accurate to atleast 1 g is needed. A device for regulating the temperature of the samples during immersion with salt solution is also necessary, besides a thermostatically controlled drying oven maintained at temperature 105°C to 110°C and having an average rate of evaporation of atleast 25 g per hour.

Procedure

Saturated solution of Sodium Sulphate (the anhydrous Na_2SO_4 or the crystalline $Na_2SO_4\ 10\ H_2O$) is prepared in water at a temperature of 25° to 30°C. In should be ensured that the solution is saturated and excess salt is present. The solution is maintained at a temperature of 27° ± 2°C and stirred at frequent intervals, until it is used. At the time of using the solution should have a specific gravity of not less than 1.151 and not greater than 1.171, and discoloured solution should not be used. It may be necessary to use not less than 420 g of anhydrous salt or 1.300 g of the crystalline decahydrate salt per litre of water.

Alternatively saturated solution of Magnesium sulphate may be prepared by dissolving either anhydrous ($MgSO_4$) or crystalline ($MgSO_4\ 7\ H_2O$) magnesium sulphate. At the time of using, the solution should have a specific gravity of not less than 1.295 and not more than 1.308. Not less than 400 g of the anhydrous salt or 1600 g of the crystalline hepta hydrate may be used per litre of water.

The specimen of coarse aggregate for the test may be prepared after removing the fraction finer than 4.75 mm IS sieve. The sample should be of such a size that it will yield not less than the following amounts of the different sizes, which should be available in amounts of 5 percent or more.

Sieve Size of Square Hole :

10 mm to 4.75 mm	—	300 g	
20 mm to 10.0 mm	—	1000 g	
consisting of 12.5 to 10 mm			33%
20 to 12.5 mm			67%

40 mm to 20 mm	— 1500 g	
consisting of 25 to 20 mm		33%
40 to 25 mm		67%
63 mm to 40 mm	— 3000 g	
consisting of 50 to 40 mm		50%
63 to 50 mm		50%
80 mm and larger sizes by 20 mm spread in sieve size, each fraction	— 3000 g	

The sample of coarse aggregate should be thoroughly washed and dried to a constant weight at 105° to 110°C and is separated to different size ranges, as given above, by sieving. The proper weight of the sample for each fraction is weighed and placed in separate containers for the test. In the case of fraction coarser than 20 mm, the particles are also counted. The samples are immersed in the prepared solution of sodium sulphate or magnesium sulphate for 16 to 18 hours in such a manner that the solution cover them to a depth of atleast 15 mm. The containers are kept covered to reduce evaporation and during the period of immersion, the temperature of the solution is maintained at 27° ± 1°C.

After the immersion period, the aggregates are removed from the solution, drained for about 15 minutes, and placed in the drying oven maintained at a temperature of 105° to 110°C. The samples are dried to a constant weight at this temperature by checking the weights after 4 hours up to 18 hours. When the successive weights differ by less than 1 g, it may be considered that constant weight has been attained and then it may be allowed to cool to room temperature. Then the aggregates are again immersed in the prepared solution for the next cycle of immersion and drying. The number of cycles of alternate immersion and drying are decided by prior aggreement before the purchase.

After completion of the final cycle, the sample is cooled washed free from the sulphate. This may be determined when there is no more reaction of the wash water with barium chloride (i.e., when there is no white precipitation when barium chloride is added to wash water, it can be said that there is no sulphate with wash water). Each fraction of the sample is then dried to constant temperature of 105°C to 110°C and weighed. Coarse aggregate fractions are sieved by IS sieves of sizes indicated below :

Size of Aggregate	*Sieve Size used to determine loss*
63 to 40 mm	31.5 mm
40 to 20 mm	16.0 mm
20 to 10 mm	8.0 mm
10 to 4.75 mm	4.0 mm

Each fraction of aggregate is examined visually to see if there is any evidence of excessive splitting, crumbling or disintegration of the grains. A combined sieve analysis of all the materials subjected to the above test cycles, may also be carried out to note the variation from the original grain size distribution of the sample.

Results

The results should be reported giving the following particulars :

(a) Type of solution used for the test.

(b) Weight of each fraction of sample before test.

(c) Material from each fraction of the sample finer than the sieve on which the fraction was retained before test expressed as a percentage by weight of the fraction.

(d) Weighted average calculated from the percentage loss for each fraction based on the grading of the sample as received for examination or, preferably on the average grading of the material from that portion of supply of which the sample is representative.

(e) In the case of particles coarser than 20 mm size before the test, the number of particles in each fraction before test and the number of particles affected, classified as to the number disintegrating, splitting crumbling cracking, flaking etc.

A recommended form for recording of data is given in Table 13.1 with illustrative test values.

TABLE 13.1

Soundness Test Data for Coarse Aggregates

Sieve size, mm		Grading of original sample percent	Weight of test fractions before test, g	Percentage passing finer sieve after test (actual percent loss)	Weighted average (corrected percent loss)
Passing	Retained				
1	2	3	4	5	$6 = 5 \times 3$
				4.8	$4.8 \times \frac{20}{100} = 0.96$
63	40	20.0	3000	8.0	3.60
40	20	45.0	1500	9.6	2.20
20	10	23.0	1000	11.2	1.34
10	4.75	12.0	300		8.10
Total		100.0	5800		

Discussion

If the samples contain less than 5 percent of any of the sizes specified under procedure that size should not be tested; but for the purpose of lcaculating the test result, it shall be considered to have the same loss in sodium sulphate or magnesium sulphate treatment as the average of the next smaller and next larger size. If one of these sizes is absent, it may be considered to have the same loss as the next larger or next smaller sizes whichever is present. When the 20 to 10 mm, 40 to 20 mm or 63 to 40 mm test samples specified cannot be prepared due to the absence of one or two sizes of aggregate shown for each, the size available may be used to prepare the sample.

Applications of Soundness Test

The soundness test is useful to assess the resistance of the aggregate to weathering. For cement concrete likely to be exposed to the action of frost, coarse and fine aggregates should pass an accelerated soundness test specified above. The limits for the test result are set by aggreement between the purchaser and the supplier. However the aggregates which fail in the accelerated soundness test, may be used if they pass a specified freeze-thaw test satisfactory to the user.

As a general guide, it may be taken that the average loss of weight after 10 cycles should not exceed 12 percent when tested with sodium sulphate and 18 percent when tested with magnesium sulphate.

Indian Roads Congress has specified 12 percent as the maximum permissible loss in soundness test after 5 cycles with sodium sulphate for the aggregates to be used in bituminous surface dressing, penetration Macadam and bituminous Macadam construction.

REFERENCES

1. Indian Standard Methods of Test for Aggregates for Concrete, IS : 2386 Part V, *Indian Standards Institution.*
2. Indian Standard Specification for Coarse and Fine Aggregates from Natural Sources for Concrete, IS : 383, *Indian Standards Institution.*
3. Recommended Practice for Bituminous Penetration Macadam, IRC : 20, *Indian Roads Congress.*
4. Tentative Specification for Single Coat Bituminous Surface Dressing, IRC : 17, *Indian Roads Congress.*
5. Tentative Specification for Bituminous Macadam (Base and Binder Course), IRC : 27, *Indian Roads Congress.*

PROBLEMS

1. What do you mean by soundness test for aggregate ?
2. What is the necessity of testing the durability of aggregate in the laboratory ?
3. What are the salts usually used in the soundness test for aggregates ?
4. Which of the two salts—Sodium sulphate or magnesium sulphate—gives a higher loss in weight after 'n' cycles in the soundness test ?

OBSERVATION SHEET

SOUNDNESS TEST

Type of reagent used :

Type of coarse aggregate sample : Number of cycles :

Sieve size, mm		Grading of original sample, percent	Weight of test fraction before test, g	Percentage passing finer sieve after test (actual percent loss)	Weighted average (corrected percent loss)
Passing	retained on				
1	2	3	4	5	6
60	40				
40	20				
20	10				
10	4.75				

Number of particles coarser than 20 mm before test			Number of particles affected, classified as to the number disintegrating, splitting, crumbling, cracking or flaking
Passing	Retained	Number before test	
40 mm	20 mm		
60 mm	40 mm		

Remarks :

14

Specific Gravity and Water Absorption Tests

The specific gravity of an aggregate is considered to be a measure of strength or quality of the material. Stones having low specific gravity are generally weaker than those with higher specific gravity values. The specific gravity test helps in the identification of stone.

Water absorption gives an idea of strength of rock. Stones having more water absorption are more porous in nature and are generally considered unsuitable unless they are found to be acceptable based on strength, impact and hardness tests.

Apparatus

The apparatus consists of the following :

(a) A balance of capacity about 3 kg, to weigh accurate to 0.5 g, and of such a type and shape as to permit weighing of the sample container when suspended in water.

(b) A thermostatically controlled oven to maintain temperature of 100° to 110°C.

(c) A wire basket of not more than 6.3 mm mesh or a perforated container of convenient size with thin wire hangers for suspending it from the balance.

(d) A container for filling water and suspending the basket.

(e) An air tight container of capacity similar to that of the basket (referred to in (c) above).

(f) A shallow tray and two dry absorbent clothes, each not less than 75 × 45 cm.

Procedure

About 2 kg of the aggregate sample is washed thoroughly to remove fines, drained and then placed in the wire basket and immersed in distilled water at a temperature between 22° and 32°C with a cover of atleast 5 cm of water above the top of the basket. Immediately after immersion the entrapped air is removed from the sample by lifting the basket containing it 25 mm above the base of the tank and allowing it to drop 25 times at the rate of about one drop per second. The basket and the aggregate should remain completely immersed in water for a period of 24 ± 1/2 hours afterwards.

The basket and the sample are then weighed while suspended in water at a temperature of 22° to 32°C. In case it is necessary to transfer the basket and the sample to a different tank for weighing, they should be jolted 25 times as described above in the new tank to remove air before weighing. This weight is noted while suspended in water = W_1 g. The basket and the aggregate are then removed from water and allowed to drain for a few minutes, after which the aggregates are transferred to one of the dry absorbent clothes. The empty basket is then returned to the tank of water, jolted 25 times and weighed in water = W_2 g.

The aggregates placed on the absorbent clothes are surface dried till no further moisture could be removed by this cloth. Then the aggregates are transferred to the second dry cloth spread in a single layer, covered and allowed to dry for at least 10 minutes until the aggregates are completely surface dry. 10 to 60 minutes drying may be needed. The aggregate should not be exposed to the atmosphere, direct sunlight or any other source of heat while surface drying. A gentle current of unheated air may be used during the first ten minutes to accelerate the drying of aggregate surface. The surface dried aggregate is then weighed = W_3 g. The aggregate is placed in a shallow tray and kept in an oven maintained at a temperature of 110°C for 24 hours. It is then removed from the oven, cooled in an air-tight container and weighed = W_4 g.

At least two tests should be carried out, but not concurrently.

Calculations

Weight of saturated aggregate suspended in water with the basket	$= W_1$ g
Weight of basket suspended in water	$= W_2$ g
Weight of saturated aggregate in water	$= (W_1 - W_2) = W_s$ g
Weight of saturated surface dry aggregate in air	$= W_3$ g
Weight of water equal to the volume of the aggregate	$= (W_3 - W_s)$ g

(1) Specific gravity $= \dfrac{\text{dry weight of aggregate}}{\text{weight of equal volume of water}}$

$$= \frac{W_4}{W_3 - W_s} = \frac{W_4}{W_3 - (W_1 - W_2)}$$

(2) Apparent specific gravity $= \dfrac{\text{dry weight of aggregate}}{\text{weight of equal volume of water excluding air voids in aggregates}}$

$$= \frac{W_4}{W_4 - W_s} = \frac{W_4}{W_4 - (W_1 - W_2)}$$

(3) Water absorption = percent by weight of water absorbed in terms oven dried weight of aggregates

$$= \frac{(W_3 - W_4)\,100}{W_4} \text{ percent}$$

Discussion

The size of the aggregates and whether it has been artificially heated should be indicated. The ISI specifies three methods of testing for the determination of the specific gravity and water absorption of aggregates, according to the size of aggregates. The three size ranges used are

(i) aggregate larger than 10 mm
(ii) between 10 mm and 40 mm, and
(iii) smaller than 10 mm.

The water absorption test does not always give reproducible results with aggregates of high porosity.

Applications of Specific Gravity and Water Absorption Tests

The specific gravity of aggregates normally used in road construction ranges from about 2.5 to 3.0 with an average value of about 2.68. Though high specific gravity of an aggregate is considered as an indication of high strength, it is not possible to judge the suitability of a sample of road aggregate without finding the mechanical properties such as aggregate crushing, impact and abrasion values.

Water absorption of an aggregate is accepted as measure of its porosity. Some times this value is even considered as a measure of its resistance to frost action, though this has not yet been confirmed by adequate research.

Water absorption value ranges from 0.1 to about 2.0 percent for aggregates normally used in road surfacings. Stones with water absorption upto 4.0 percent have been used in base courses. Generally a value of less than 0.6 percent is considered desirable for surface course, though slightly higher values are allowed in bituminous constructions. Indian Roads Congress has specified the maximum water absorption value as 1.0 percent for aggregates used in bituminous surface dressing and built-up spray grout.

REFERENCES

1. Indian Standard Methods of Test for Aggregate for Concrete, IS : 2386, Part III, *Indian Standards Institution.*
2. Soil Mechanics for Road Engineers, *D.S.I.R.*, H.M.S.O., London.
3. Bituminous Materials in Road Construction, *D.S.I.R.*, H.M.S.O., London.
4. Tentative Specification for Bituminous Surface Dressing, (Single, Two-coats and Pre-coated types), IRC : 17, 23 & 48, *Indian Roads Congress.*
5. Tentative Specification for Built-up Spray Grout, IRC : 47, *Indian Roads Congress.*

PROBLEMS

1. Discuss the importance of specific gravity test on road aggregates?
2. Define true and apparent specific gravity of aggregates?
3. What is the significance of water absorption test on aggregates?

OBSERVATION SHEET

DETERMINATION OF SPECIFIC GRAVITY AND WATER ABSORPTION

(i) Size of the aggregates = (ii) Aggregate Type =

Details	Test number 1	Test number 2	Mean value
1. Weight of saturated aggregate and basket in water $= W_1$ g			
2. Weight of basket in water $= W_2$ g			
3. Weight of saturated surface dry aggregates in air $= W_3$ g			
4. Weight of oven dried aggregates in air $= W_4$ g			
5. Specific gravity $= \dfrac{W_4}{W_3 - (W_1 - W_2)}$			
6. Apparent specific gravity $= \dfrac{W_4}{W_4 - (W_1 - W_2)}$			
7. Water absorption $= \dfrac{(W_3 - W_4)\,100}{W_4}$ percent			

(i) Mean value of specific gravity =
(ii) Mean value of apparent specific gravity =
(iii) Mean value of water absorption =

Report on quality of stone

Remarks :

15 Shape Tests

INTRODUCTION

The particle shape of aggregates is determined by the percentages of flaky and elongated particles contained in it. In the case of gravel it is determined by its angularity number. For base course and construction of bituminous and cement concrete types, the presence of flaky and elongated particles are considered undesirable as they may cause inherent weakness with possibilities of breaking down under heavy loads. Rounded aggregates are preferred in cement concrete road construction as the workability of concrete improves. Angular shape of particles are desirable for granular base course due to increased stability derived from the better interlocking. When the shape of aggregates deviates more from the spherical shape, as in the case of angular, flaky and elongated aggregates, the void content in an aggregate of any specified size increases and hence the grain size distribution of a graded aggregate has to be suitably altered in order to obtain minimum voids in the dry mix or the highest dry density. The angularity number denotes the void content of single sized aggregates in excess of that obtained with spherical aggregates of the same size. Thus angularity number has considerable importance in the gradation requirements of various types of mixes such as bituminous concrete and soil-aggregate mixes.

Thus evaluation of shape of the particles, particularly with reference to flakiness, elongation and angularity is necessary.

FLAKINESS INDEX

The *flakiness index* of aggregates is the percentages by weight of particles whose least dimension (thickness) is less than three-fifths (0.6) of their mean dimension. The test is not applicable to sizes smaller than 6.3 mm.

Apparatus

The apparatus consists of a standard thickness gauge shown in Figure 15.1, IS sieves of sizes 63, 50, 40,

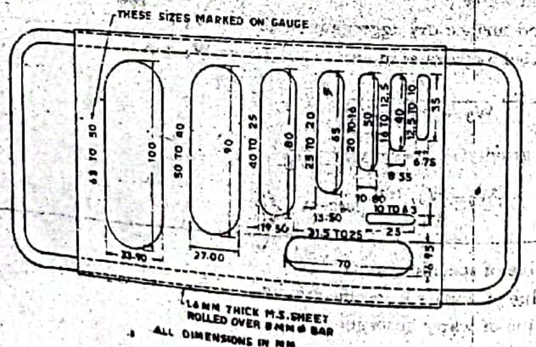

Figure 15.1 Thickness Gauge

31.5, 25, 20, 16, 12.5, 10 and 6.3 mm and a balance to weigh the samples.

Procedure

The sample is sieved with the sieves mentioned in Table 15.1. A minimum of 200 pieces of each fraction to be tested are taken and weighed = W_1 g. In order to separate flaky materials, each fraction is then gauged for thickness on a thickness gauge shown in Figure 15.1 or in bulk on sieves having elongated slots. The width of the slot used should be of the dimensions specified in column (3) of Table 15.1 for the appropriate size of material. The amount of flaky material passing the gauge is weighed to an accuracy of at least 0.1 percent of the test sample.

TABLE 15.1

Dimensions of Thickness and Length Gauges

Size of aggregate		(a) Thickness gauge (0.6 times the mean sieve), mm	(b) Length gauge (1.8 times the mean sieve), mm
Passing through IS sieve mm	Retained on IS sieve, mm		
1	2	3	4
63.0	50.0	33.90	—
50.0	40.0	27.00	81.0
40.0	25.0	19.50	58.5
31.5	25.0	16.95	—
25.0	20.0	13.50	40.5
20.0	16.0	10.80	32.4
16.0	12.5	8.55	25.6
12.5	10.0	6.75	20.2
10.0	6.3	4.89	14.7

Calculation and Result

In order to calculate the flakiness index of the entire sample of aggregates first the weight of each fraction of aggregate passing and retained on the specified set of sieves is noted. As an example let 200 pieces of the aggregate passing 50 mm sieve and retained on 40 mm sieve be = W_2 g. Each of the particle from this fraction of aggregate is tried to be passed through the slot of the specified thickness of the thickness gauge; in this example the width of the appropriate gauge of the thickness gauge is $= \frac{(50+40)}{2} \times 0.6 = 27.0$ mm gauge. Let the weight of the flaky material passing this gauge be w_2 g. Similarly the weights of the fractions passing and retained the specified sieves, W_1, W_2, W_3 etc. are weighed and the total weight $W_1 + W_2 + W_3 + \ldots = W$ g is found. Also the weights of material passing each of the specified thickness gauge are found $= w_1, w_2, w_3 \ldots$ and the total weight of material passing the different thickness gauges $= w_1 + w_2 + w_3 + \ldots = w$ g is found. Then the flakiness index is the total weight of the flaky material passing the various thickness gauges expressed as a percentage of the total weight of the sample gauged.

$$\text{Flakiness Index} = \frac{(w_1 + w_2 + w_3 + \ldots)100}{W_1 + W_2 + W_3 + \ldots} \text{ percent} = 100 \frac{w}{W} \text{ percent}$$

ELONGATION INDEX

The elongation index of an aggregate is the percentage by weight of particles whose greatest dimension (length) is greater than one and four fifth times (1.8 times) their mean dimension. The elongation test is not applicable to sizes smaller than 6.3 mm.

Apparatus

The apparatus consists of the length gauge shown in Figure 15.2, sieves of the sizes specified in Table 15.1 and a balance.

Procedure

The sample is sieved through the IS sieves specified in Table 15.1. A minimum of 200 pieces of each fraction is taken and weighed. In order to separate elongated material, each fraction is then gauged individually for length in a length gauge (See Figure 15.2). The gauge lengths used should be those specified in column 4 of the Table for the appropriate material. The pieces of aggregates from each fraction tested which could not pass through the specified gauge length with its long side are elongated particles and are collected separately to find the total weight of aggregates retained on the length gauge from each fraction. The total amount of elongated material retained by the length gauge are weighed to an accuracy of atleast 0.1 percent of the weight of the test sample.

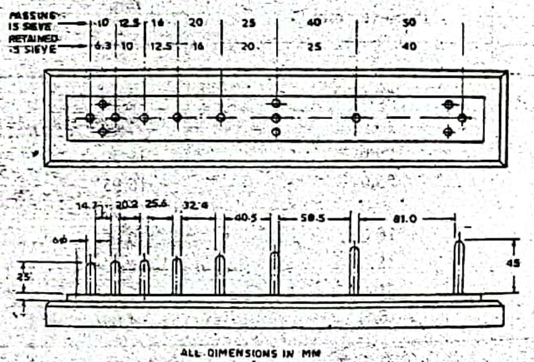

Figure 15.2 Length Gauge

Calculation and Result

In order to calculate the elongation index of the entire sample of aggregates the weight of aggregate which is retained on the specified gauge length from each fraction is noted. As an example, let 200 pieces of the aggregate passing 40 mm sieve and retained 25 mm sieve weigh W_1 g. Each piece of these are tried to be passed through the specified gauge length of the length gauge, which in this example is $= \frac{(40+25)}{2} \times 1.8 = 58.5$ mm with its longest side and those elongated pieces which do not pass the gauge are separated and the total weight determined $= w_1$ g. Similarly the weight of each fraction of aggregate passing and retained on specified sieves sizes are found, $W_1, W_2, W_3, \ldots$ and the total weight of sample determined $= W_1 + W_2 + W_3 + \ldots = W$ g. Also the weight of material from each fraction retained on the specified gauge length are found $= x_1, x_2, x_3 \ldots$ and the total weight retained determined $= x_1 + x_2 + x_3 + \ldots = X$ g.

The elongation index is the total weight of the material retained on the various length gauges, expressed as a percentage of the total weight of the sample gauged.

$$\text{Elongation Index} = \frac{(x_1+x_2+x_2 \ldots)\,100}{W_1+W_2+W_3+\ldots} = 100\,\frac{X}{W} \text{ percent}$$

ANGULARITY NUMBER

Based on the shape of the aggregate particle, stones may be classified as rounded, angular and flaky. Angular particles possess well defined edges formed at the intersection of roughly plane faces and are commonly found in aggregates prepared by crushing of rocks. Since weaker aggregates may be crushed during compaction, the angularity number does not apply to any aggregate which breaks down during compaction.

Angularity or absence of the rounding of the particles of an aggregate is a property which is of importance because it affects the ease of handling a mixture of aggregate and binder or the workability of the mix. The determination of angularity number of an aggregate is essentially a laboratory method intended for comparing the properties of different aggregates for mix design purposes and for deciding their gradation requirements.

The degree of packing of particles of single sized aggregate depends on the shape and angularity of the aggregates. If a number of single sized spherical particles are packed together in the densest form, the total volume of solids will be 67 percent and the volume of voids 33 percent of the total volume. However if the shape of the particles of the same size deviates from the spherical shape to irregular or angular shape, when they are densely packed, the volume of solids decreases resulting in an increase in the volume of voids. Hence the angularity of the aggregate can be estimated from the properties of voids in a sample of aggregates compacted in a particular manner. The angularity number of an aggregate is the amount by which the percentage voids exceeds 33 after being compacted in a prescribed manner. The angularity number is found from the expression, (67 minus the percent solid volume). Here the value 67 represents the percentage volume of solids of most rounded gravel which would have 33 percent voids.

Apparatus

The apparatus consists of (a) a metal cylinder closed at one end and of about 3 litre capacity, the diameter and height of this being approximately equal, ie., about 15.64 cm dia. × 15.64 cm height.

(b) A metal tamping rod of circular cross section, 16 mm in diameter and 60 cm in length, rounded at one end.

(c) A metal scoop of about one litre heaped capacity of size 20×10×5 cm, and

(d) A balance of capacity 10 kg to weigh up to 1.0 g

Procedure

The cylinder is calibrated by determining the weight of water at 27°C required to fill it, so that no meniscus is present above the rim of the container. The amount of aggregate available should be sufficient to provide, after separation on the appropriate pair of sieves, atleast 10 kg of the predominant size, as determined by the sieve analysis on the 20, 16, 12.5, 10, 6.3 and 4.75 mm IS sieves. The test sample should consist of aggregate retained between the appropriate pair of IS sieves having square holes from the following sets :

20 and 16 mm , 16 and 12.5 mm , 12.5 and 10 mm , 10 and 6.3 mm , 6.3 and 4.75 mm

In case aggregate larger than 20 mm sieve is used for the test, the volume of the cylinder should be greater than 3 litres, but when aggregates smaller than 4.75 mm size are used, a smaller cylinder may be

The procedure is the same for each of these except that the amount of compactive effort given by (weight of the tamping rod × height of fall × number of blows) should be proportional to the volume of the cylinder.

The sample of single size aggregate retained between the specified pair of sieves is dried in an oven at a temperature 100° to 110°C for 24 hours and cooled in an air tight container prior to testing. The scoop is filled and heaped to overflowing with the aggregate, which is placed in the cylinder by allowing it to slide gently off the scoop from the lowest possible height. The aggregates in the cylinder are subjected to 100 blows of the tamping rod at a rate of about 2 blows per second. Each blow is applied by holding the rod vertical with its rounded end 5 cm above the surface of the aggregate and releasing it so that it falls vertically and no force is applied to the rod. The 100 blows should be distributed evenly over the surface of the aggregates.

The process of filling and tamping is repeated exactly as described above with a second and third layer of aggregates. The third layer should contain only the aggregate required to just fill up the cylinder level before tamping. After the third layer is tamped, the cylinder is filled to over flowing, and the aggregates are struck off level with the top using tamping rod as a straight edge.

Individual pieces of aggregate are then added and rolled in to the surface by rolling the tamping rod across the upper edge of the cylinder, and this finishing process is continued as long as the aggregate do not lift the rod off the edge of the cylinder on either side, during rolling. The aggregate should not be pushed in or forced down and no downward pressure should be applied to the tamping rod, which is only rolled in contact with the top of the cylinder, on both sides.

The aggregate with cylinder is then weighed to the nearest 5 g. Three separate determinations are made and the mean weight of the aggregate in the cylinder is calculated. If the result of any one of the determination differs from the mean by more than 25 g, three additional determinations are immediately made on the same material and the mean of all the six determinations is calculated.

Calculation and Results

The angularity number is calculated from the formula :

$$\text{Angularity number} = 67 - 100\,W/CG$$

Where W = mean weight of aggregates in the cylinder, g

C = Weight of water required to fill the cylinder, g

G = Specific gravity of aggregate.

The angularity number is expressed to the nearest whole number.

Discussion

The shape tests give only a rough idea of the relative shapes of the aggregates. Particular care has to be taken while carrying out the test for angularity number.

Application of Shape Tests

In pavement construction flaky and elongated particles are to be avoided, particularly in surface course. If flaky and elongated aggregates are present in appreciable proportions, the strength of the pavement layer would be adversely affected due to possibility of breaking down under loads. In cement concrete the workability is also reduced. However, the reduction in strength in cement concrete depends on the cement content and water-cement ratio.

Indian Roads Congress has recommended the maximum allowable limits of flakiness index values for various types of construction, as given in Table 15.2.

TABLE 15.2

Maximum Allowable Flakiness Index of Aggregates in Different Types of Pavement Construction

Serial No.	Type of pavement construction	Maximum limit of Flakiness Index, %
1	Bituminous Carpet	30
2	(i) Bituminous/Asphaltic concrete	
	(ii) Bituminous surface dressing (single coat, two coats and pre-coated)	
	(iii) Bituminous penetration macadam	
	(iv) Built-up spray grout	25
3	(i) Bituminous macadam	
	(ii) Water bound macadam, base and surfacing courses	15

Though elongated shape of the aggregates also affects the compaction and the construction of pavements, there are no specified limits of elongation index values as in the case of flakiness index for different methods of pavement construction.

The angularity number measures the percent voids in excess of 33 percent which is obtained in the case of the most rounded gravel particles. The angularity number of aggregates generally ranges from zero for highly rounded gravel to about 11 for freshly crushed angular aggregates. Slightly higher values of angularity number also may be obtained in the case of highly angular and flaky aggregates. Thus higher the angularity number, more angular and less workable is the aggregate mix. In cement concrete mix, rounded aggregates may be preferred because of better workability, lesser specific surface and higher strength for a particular cement content. But in flexible pavement construction methods using hard aggregates such as the bituminous construction methods, water bound macadam, etc., angular aggregates are preferred because of higher stability due to better interlocking and friction. However in dense bituminous mixes such as the bituminous concrete, the gradation requirement may have to be suitably modified during mix design in the case of aggregates with high angularity number so as to obtain a well designed mix.

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1. Indian Standard Methods of test for Aggregates for Concrete, IS : 2386, part I, *Indian Standards Institution.*
2. Bituminous Materials in Road Construction, *D.S.I.R.*, H.M.S.O., London.
3. Properties of Concrete *A.M. Neville*, Sir Isaac Pitman & Sons Ltd., London.
4. Concrete-Properties and Manufacture, *T.N.W. Akroyd*, Pergamon Press.
5. A Simplified Approach for Mix Design of Bituminous Concrete, *Rajagopal, Veeraragavan* and *Justo*, International Conf. on Aggregates, Nice, May 1984.

PROBLEMS

1. What is the significance of shape of aggregate in pavement construction ?
2. Discuss the effects of flaky and elongated aggregate in road construction ?
3. Explain what is meant by flaky and elongated particles ?

4. Explain Flakiness Index. How is it found ?
5. What is Elongation Index ? How is it determined in the laboratory ?
6. Discuss the advantages and limitations of rounded and angular aggregates in different type of pavements.
7. Explain Angularity Number. How is it found ?
8. What are the applications of shape tests ?

OBSERVATION SHEET

Flakiness Index and Elongation Index

General description of the aggregate

Size of aggregate: Passing through IS sieve, mm	Size of aggregate: Retained on IS sieve, mm	Weight of the fraction consisting of atleast 200 pieces, g	Thickness gauge size, mm	Weight of aggregate in each fraction passing thickness gauge, g	Length gauge size, mm	Weight of aggregates in each fraction retained on length gauge, g
1	2	3	4	5	6	7
63	50	W_1 =	23.90	w_1 =	—	—
50	40	W_2 =	27.00	w_2 =	81.0	x_1 =
40	25	W_3 =	19.50	w_3 =	58.0	x_2 =
31.5	25	W_4 =	16.95	w_4 =	—	—
25	20	W_5 =	13.50	w_5 =	40.5	x_3 =
20	16	W_6 =	10.80	w_6 =	32.4	x_4 =
16	12.5	W_7 =	8.55	w_7 =	25.6	x_5 =
12.5	10.0	W_8 =	6.75	w_8 =	20.2	x_6 =
10.0	6.3	W_9 =	4.89	w_9 =	14.7	x_7 =
Total		W =		w =		x =

$$\text{Flakiness Index} = \frac{(w_1+w_2+w_3+\ldots)}{(W_1+W_2+W_3+\ldots)}\,100 \text{ percent} = \frac{100\,w}{W} \text{ percent} =$$

$$\text{Elongation Index} = \frac{(x_1+x_2+x_3+\ldots)}{(W_1+W_2+W_3+\ldots)}\,100 \text{ percent} = \frac{100\,x}{W} \text{ percent} =$$

ANGULARITY NUMBER

Weight of water filling the cylinder = C g =

Specific gravity of the aggregate = G =

Particulars	Trial number							
	1	2	3	Mean	4	5	6	Mean
Weight of aggregate filling the cylinder to the nearest five grams, g								

Mean weight of aggregate filling the cylinder, Wg =

$$\text{Angularity Number} = 67 - \frac{100\,W}{CG}$$

Remarks

Stripping Value of Road Aggregates

INTRODUCTION

Bitumen and tar adhere well to all normal types of aggregates provided they are dry and are not exceptionally dusty. The process of binding is controlled largely by the viscosity of the binder. When the viscosity of the binder is high, coating of aggregates by the binder is slower. In the absence of water there is practically no adhesion problem in bituminous road construction. Two problems are observed due to the presence of water. First, if aggregate is wet and cold it is normally not possible to coat with a bituminous binder. This problem can be dealt with by removing the water film on the aggregate by drying and by increasing the mixing temperature. Second problem is *stripping* of coated binder from the aggregate due to the presence of water. This problem of stripping is experienced only with bituminous mixtures which are permeable to water.

The stripping is due to the fact that those aggregates have geater affinity towards water than with bituminous binders, and this displacement depends on the physico-chemical forces acting on the system. Most road stones have surfaces that are electrically charged. As an example silica, a common constituent of igneous rock possesses a weak negative charge and hence these have a greater attraction with the polar liquid water than with bituminous binders having little polar activity.

In order to study the suitability of road aggregates for bituminous road construction, it is desirable to study the displacement characteristics of the binders from the aggregates by water. Several laboratory tests have been developed to arbitrarily determine the adhesion of bituminous binder to an aggregate in the presence of water. These tests may be classified into six types (i) Static immersion tests (ii) Dynamic immersion tests (iii) Chemical immersion tests (iv) Immersion mechanical tests (v) Immersion trafficking tests and (vi) Coating tests.

The static immersion test is very commonly used as it is easy and simple. The principle of this type of test is immersing aggregates coated with the binder in water, and estimating the degree of stripping.

A method of assessing the stripping value of road aggregates coated with bitumen or tar has been recommended by the Central Road Research Institute (CRRI) New Delhi. The method of test for determination of stripping value of road aggregate has also been standardised by the ISI and this test has been given here. Also the stripping test method developed by the Road Research Laboratory (RRL) England has been given in this chapter.

STRIPPING VALUE OF ROAD AGGREGATES

Apparatus

Thermostatically controlled water bath, beaker, mixer etc.

Procedure

This method covers the procedure for determining the stripping value of aggregates by static immersion method, when bitumen and tar binders are used. 200 g of dry and clean aggregates passing 20 mm IS sieve and retained on 12.5 mm sieve are heated up to 150°C when these are to be mixed with bitumen and the aggregates are heated upto 100°C when these are to be mixed with tar. Five percent by weight of bitumen binder is heated to 160°C (110°C in the case of tar binder). The aggregate and binder are mixed thoroughly till they are completely coated and the mixture is transferred to a 500 ml beaker and allowed to cool at room temperature for about two hours. Distilled water is then added to immerse the coated aggregates. The beaker is covered and kept in a water-bath maintained at 40°C, taking care that the level of water in the water-bath is atleast half the height of the beaker. After 24 hours the beaker is taken out, cooled at room temperature and the extent of stripping is estimated visually while the specimen is still under water.

The stripping value is the ratio of the uncovered area observed visually to the total area of aggregates in each test, expressed as a percentage. The mean of three results is reported as stripping value of the tested aggregates and is expressed as the nearest whole number.

STATIC IMMERSION TEST BY R.R.L.

Apparatus

Thermostatically controlled drying ovens capable of maintaining constant temperature between 26°C and 120°C, Balance of capacity of 1 kg and accuracy 0.1 g, mixing pan spatula, jars etc.

Procedure

Aggregates passing 19 mm sieve and retaind on 12.7 mm sieve is washed thoroughly with water rinsed in distilled water and dried in an oven at 120°C for two hours. For each test 500 g each of coated mixture is prepared. When bitumen is used as a binder, 3.5 percent of the total weight is taken and so 482.5 g of aggregate is mixed with 17.5 g of bitumen to get a total weight of 500 g of the coated mixture. When tar is used as the binder 4.25 percent of total weight is taken for mixing and therefore 478.75 g of aggregates is mixed with 21.25 g of tar to make 500 g of mixture. When using cutback bitumen or tar, aggregates at 60°C and binder at 100°C are used. Mixing is carried out by hand in a suitable mixing pan using a spatula and mixing is continued for 5 minutes at approximately 60 turns per minute. The uncoated areas of aggregates which are found at the end may be touched up. In order to make up the loss of binder due to the some adhering to the pan and spatula, the first sample of coated mix may be rejected without testing and the process of mixing the weighed quantities of aggregates and binder is repeated for preparing test specimens.

The sample is divided into two approximately equal parts after mixing and each is placed in a 500 cc glass jar allowed to stand at room temperature for 3 hours. One of two jars containing the mixture is then filled with distilled water boiled previously and brought to a temperature of 25°C. The other jar with mixture is filled with distilled water previously boiled and brought to a temperature of 40°C.

The jars are then covered and maintained at 25°C and 40°C respectively until the test is completed. Observations are made at 1, 3, 24 and 48 hours after the water is added. In every case the percentage of stone surface that remains coated after the above times are visually estimated. At least three opinions may be taken in estimating the percentage aggregates coated and the average value is taken.

Results

The result is reported as the percentage of stone surface that remains coated after the specified periods the mean value of at least three visually estimated values, being rounded off to the nearest 5 percent.

Discussion

The visual assessment of stripping leads to poor reproducibility. But still the test gives an idea of how a mixture of aggregates and binder may behave in presence of water.

Attempts have been made to improve the test by adopting other methods of assessing the degree of stripping. One of the approaches is by measuring the quantity of light reflected by the sample of coated

aggregates before and after immersion in water. The degree of stripping may also be mechanically measured indirectly by measuring the change in a mechanical property like compressive strength. The reduction in strength gives an indication of the extent of damage by the water that has occurred.

It is reported that the adhesion test results are not always reliable from the point of view of field performance.

Central Road Research Institute (C.R.R.I.), New Delhi has recommended a stripping test for aggregates. Indian Roads Congress has specified the allowable limits of this stripping test results for the aggregates used in bituminous road construction.

Application of Stripping Test

Some types of aggregates have lesser affinity with bitumen in comparison with water and hence stripping of the bituminous binder occurs when the mix is immersed in water. The problem of stripping in coated aggregates is not so amenable to theoretical treatment. Thus an adhesion test such as the simple static immersion test or the stripping test would be suitable to assess whether the binder would adhere to the aggregate when immersed in water. Several anti-stripping agents are available, which when used with the bituminous mix reduce the stripping.

Indian Road Congress has specified the maximum stripping value as 25 percent for aggregates to be used in bituminous constructions like surface dressing, penetration macadam, bituminous macadam and carpet.

REFERENCES

1. Bituminous Materials in Road Construction, *D.S.I.R.*, H.M.S.O., London.
2. Tentative Specification for Bituminous Surface Dressing, IRC : 17, 23, 48; *Indian Roads Congress.*
3. Recommended Practice for Bituminous Penetration Macadam (Full Grout), IRC : 20, *Indian Roads Congress.*
4. Tentative Specification for Bituminous Macadam (Base and Binder Course). IRC : 27, *Indian Roads Congress.*
5. Method of Test for Determination of Stripping Value of Road Aggregates, IS : 6241, *Indian Standards Institution.*

PROBLEMS

1. Explain 'stripping ?
2. Discuss the significance of adhesion test on aggregates?
3. Briefly explain the static immersion test ?
4. Outline IS C.R.R.I. stripping test ?
5. What do you understand by anti-stripping agents?
6. What are the uses, applications and limitations of adhesion tests?
7. Outline the method of Static Immersion Test suggested by the T.R.R.L., London?

OBSERVATION SHEET

STRIPPING TEST

(i) Type of aggregate =
(ii) Type of binder =
(iii) Percentage binder used =
(iv) Total weight of aggregate =
(v) Total weight of binder =
(vi) Temperature of water-bath =

Observation Number	Stripping, percentage
1	
2	
3	
Average value	

Remarks :

Part III

Tests on Bituminous Materials

Introduction

Bituminous binders used in pavement construction works include both bitumen and tar. Bitumen is a petroleum product obtained by the distillation of petroleum crude where-as road tar is obtained by the destructive distillation of coal or wood. Both bitumen and tar have similar appearance as both are black in colour, both these materials can be used for pavement works, but they have different characteristics.

The bitumen is brought to sufficient fluidity or viscosity before use in pavement construction by any one of the following three methods :

(i) by heating, in the form of hot bitumen binder.

(ii) by disolving in light oils, in the form of cutback bitumen.

(iii) by dispersing bitumen in water, in the form of bituminous emulsion.

Bituminous binders are very commonly used in surface course of pavements; they are also used in the base course of flexible pavements to withstand relatively adverse conditions of traffic and climate. These materials are also used in soil-bitumen stabilization and to prepare sealer materials for filling the joints in cement concrete pavements.

Bitumen is available in a variety of types and grades. The grades of bitumen used for pavement construction work of roads and air-fields are called paving grades and those used for water proofing of structures, industrial floors, etc. are called industrial grades. The paving bitumen available in India is classified into two categories :

(i) paving bitumen from Assam petroleum, denoted as A-type and designated as A 35 grade, A 90 grade, etc.

(ii) paving bitumen from other source denoted as S-type and designated as S 35 grade, A 90 grade, etc.

There are a number of tests to assess the properties of bituminous materials. Standard agencies in various countries like the ISI in India have specifield the test details and requirements of the different materials like bitumen, cutback, emulsion and tar.

The common tests to assess the requirements of paving bitumen are the penetration test, ductility test and the softening point test; flash and fire point tests and the specific gravity tests are also needed. Additional tests like the matter soluble in carbondisulphide, loss on heating and penetration tests on residue may also be carried out. The viscosity test by orifice viscometer is important for testing tar and cutback.

Penetration Test

INTRODUCTION

The consistency of bituminous materials vary depending upon several factors such as constituents, temperature, etc. At temperature ranges between 25 and 50°C most of the paving bitumen grades remain in semi-solid or in plastic states and their viscosity is so high that they do not flow as liquid. But the viscosity of most of the tars and cutbacks are sufficiently low at this temperature range to permit these bituminous materials to be in liquid state, enabling some of the grades to be mixed with aggregates even without heating.

Determination of absolute viscosity of bituminous materials is not so simple. Therefore the consistency of these materials are determined by indireet methods; the consistency of bitumen is determined by penetration test which is a very simple test; the viscosity of tars and cutback bitumens are determined indirectly using an orifice viscometer in terms of time required for a specified quantity of material to flow through an orifice. There is a certain range of consistency of bituminous materials, where-in the material is too soft for penetration test, but the viscosity is so high that the material can not flow through the orifice of the viscometer; the consistency of such materials is measured by 'float test'.

Various types and grades of bituminous materials are available depending on their origin and refining process. The penetration test determines the consistency of these materials for the purpose of grading them, by measuring the depth (in units of one tenth of a millimeter or one hundredth of a centimeter) to which a standard needle will penetrate vertically under specified conditions of standard load, duration and temperature. Thus the basic principle of the penetration test is the measurement of the penetration (in units of one tenth of a mm) of a standard needle in a bitumen sample maintained at 25°C during five seconds, the total weight of the needle assembly being 100 g. The softer the bitumen, the greater will be the penetration.

The penetration test is widely used world over for classifying the bitumen into different grades. The ISI has standardised the penetration test equipment and the test procedure. Even though it is recognised that the empirical tests like penetration, softening point etc. can not fully qualify the paving binder for its temperature susceptibility characteristics, the simplicity and quickness of operation of this test can not be ignored for common use. The concept of the penetration test and the test set up are illustrated in Figure 17.1.

Apparatus

It consists of items like container, needle, water bath penetrometer, stop watch etc. Following are the standard specifications as per ISI for the above apparatus.

(a) Container : A flat bottomed cylindrical metallic container 55 mm in diameter and 35 mm or 57 mm in height.

(b) Needle : A straight, highly polished cylindrical, hard steel needle with conical end, having the shape and dimensions as given in Figure 17.2. The needle is provided with a shank approximately 3.0 mm in diameter into which it is immovably fixed.

(c) Water-bath : A water bath is maintained at 25 ± 1°C containing not less than 10 litres of water, the sample is immersed to depth not less than 100 mm from the top and supported on a perforated shelf not less than 50 mm from the bottom of the bath.

(d) Penetrometer : It is an apparatus which allows the needle assembly of gross weight 100 g to penetrate without appreciable friction for the desired duration of time. The dial is accurately calibrated to give penetration value in units of one tenth of a mm. Electrically operated automatic penetrometers are also available. Typical sketch of penetrometer is shown in Figure 17.3.

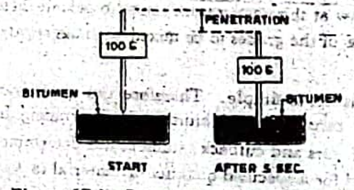


Figure 17.1 Penetration Test Concept

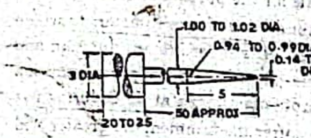


Figure 17.2 Penetration Needle

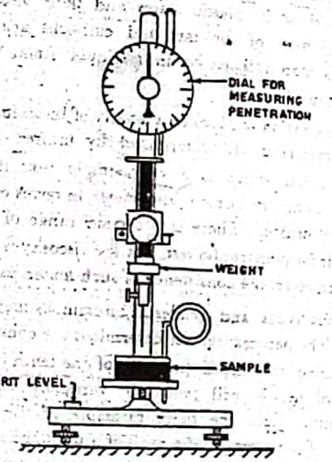


Figure 17.3 Penetrometer

(e) Transfer tray : A small tray which can keep the container fully immersed in water during the test.

Procedure

The bitumen is softened to a pouring consistency between 75° and 100°C above the approximate temperature at which bitumen softens. The sample material is thoroughly stirred to make it homogenous and free from air bubbles and water. The sample material is then poured into the containers to a depth at least 15 mm more than the expected penetration. The sample containers are cooled in atmosphere of temperature not lower than 13°C for one hour. Then they are placed in temperature controlled water bath at a temperature of 25°C for a period of one hour.

The sample container is placed in the transfer tray with water from the water bath and placed under the needle of the penetrometer. The weight of needle, shaft and additional weight are checked. The total weight of this assembly should be 100 g. Using the adjusting screw, the needle assembly is lowered and the tip of the needle is made to just touch the top surface of the sample; the needle assembly is clamped in this position. The contact of the tip of the needle is checked using the mirror placed on the rear of the needle. The initial reading of the penetrometer dial is either adjusted to zero or the initial reading is taken before releasing the needle. The needle is released exactly for a period of 5.0 secs. by pressing the knob and the final reading is taken on the dial. At least three measurements are made on this sample by testing at distance of not less than 10 mm apart. After each test the needle is disengaged and cleaned with benzene and carefully dried. The sample container is also transferred in the water bath before next

testing is done so as to maintain a constant temperature of 25°C. The test is repeated with sample in the other containers.

Results

The difference between the initial and final penetration readings is taken as the penetration value. The mean value of three consistent penetration measurements is reported as the penetration value. It is further specified by ISI that results of each measurement should not vary from the mean value reported above by more than the following :

Penetration grade	Repeatability
0—80	4 percent
80—225	5 percent
Above 225	7 percent

Discussion

It may be noted that the penetration value is influenced by any inaccuracy as regards :

(i) pouring temperature
(ii) size of needle
(iii) weight placed on the needle
(iv) test temperature
(v) duration of releasing the penetration needle

It is obvious to obtain high values of penetration if the test temperature and/or weight (place over the needle) are/is increased. Higher pouring temperatures than that specified may result in hardening of bitumen and may give lower penetration values. Higher test temperatures give considerably higher penetration values. The duration of releasing the penetration needle should be exactly 5.0 secs. It is also necessary to keep the needle clean before testing in order to get consistent results. The penetration needle should not be placed closer than 10 mm from the side of the dish.

Applications of Penetration Test

Penetration test is the most commonly adopted test on bitumen to grade the material in terms of its hardness. Depending upon the climatic condition and type of construction, bitumens of different penetration grades are used. 80/100 bitumen denotes that the penetration value ranges between 80 and 100. The penetration values of various types of bitumen used in pavement construction in this country range between 20 and 225. For bituminous macadam and penetration macadam, Indian Roads Congress suggests bitumen grades 30/40, 60/70 and 80/100. In warmer regions lower penetration grades are preferred and in colder regions bitumen with higher penetration values are used.

The penetration test is not intended to estimate the consistency of softer materials like cutback or tar, which are usually graded by a viscosity test in an orifice viscometer.

The Indian Standards Institution has classified paving bitumen available in this country into the following six categories depending on the penetration values. Grades designated 'A' (such as A 35) are from Assam Petroleum and those designated 'S' (such as S 35) are from other sources.

Bitumen grade	A 25	A 35 & S 35	A 45 & S 45	A 65 & S 65	A 90 & S 90	A 200 & S 200
Penetration value	20 to 30	30 to 40	40 to 50	60 to 70	80 to 100	175 to 225

REFERENCES

1. Indian Standard Methods for Testing Tar and Bitumen, Determination of Penetration, IS : 1203, *Indian Standards Institution.*
2. Bituminous Road Construction, *Burmah Shell*
3. Asphalts, *ESSO*
4. Bituminous Materials in Road Construction, *D.S.I.R.*, H.M.S.O., London
5. Recommended Practice for Bituminous Penetration Macadam, (Full Grout); *Indian Roads Congress.*
6. Indian Standard Specification for Paving Bitumen, IS : 73-1961, *Indian Standards Institution.*

PROBLEMS

1. How is penetration value of bitumen expressed ?
2. What are the standard load, time and temperature specified for penetration test.
3. Briefly outline the penetration test procedure.
4. What do you understand by 80/100 bitumen ?
5. What are the effects of : (i) higher test temperature (ii) higher pouring temperature (iii) exposed bitumen, on penetration test results.

OBSERVATION SHEET
PENETRATION TEST

(i) pouring Temperature, °C
(ii) Period of cooling in atmosphere, minutes =
(iii) Room temperature, °C =
(iv) Period of cooling in water bath, minutes =
(v) Actual test temperature, °C =

Readings	Sample No.				Sample No.			
	Test 1	Test 2	Test 3	Mean value	Test 1	Test 2	Test 3	Mean value
Penetrometer dial reading (i) initial								
(ii) final								
Penetration value								
Repeatability, percent								

Mean Penetration value =

Remarks

18

Ductility Test

INTRODUCTION

In the flexible pavement construction where bitumen binders are used, it is of significant importance that the binders form ductile thin films around the aggregates. This serves as a satisfactory binder in improving the physical interlocking of the aggregates. The binder material which does not possess sufficient ductility would crack and thus provide pervious pavement surface. This in turn results in damaging effect to the pavement structure. It has been stated by some agencies that the penetration and ductility properties, go together; but depending upon the chemical composition and the type of crude source of the bitumens, sometimes it has been observed that the above statement is incorrect. It may hence be mentioned that the bitumen may satisfy the penetration value, but may fail to satisfy the ductility requirements. Bitumen paving engineer would however want that both test requirements are satisfied in the field jobs. Penetration or ductility can not in any case replace each other. The ductility is expressed as the distance in centimeters to which a standard briquette of bitumen can be stretched before the thread breaks. The test is conducted at 27 ± 0.5°C and a rate of pull of 50 ± 2.5 mm per minute. The test has been standardized by the ISI. The ductility test concept is shown in Figure 18.1.

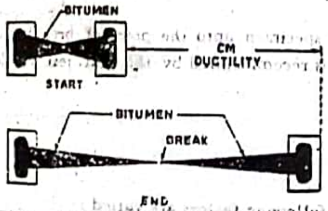


Figure 18.1 Ductility Test Concept

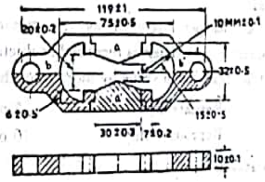


Figure 18.2 Briquette Mould

Apparatus

The ductility test apparatus consists of items like sample (briquette) moulds water bath square-end trowel or putty knife sharpened on end and ductility machine. Following are standard specifications as per the ISI for the above items :

(a) *Briquette mould :* Mould is made of brass metal with shape and dimensions as indicated in Figure 18.2. Both ends called clips possess circular holes to grip the fixed and movable ends of the testing machine. Side pieces when placed together form the briquette of the following dimensions :

Length	75 mm
Distance between clips ...	30 mm
Width at mouth of clip ...	20 mm
Cross section at minimum width ...	10 mm × 10 mm

(b) *Ductility machine :* It is equipment which functions as constant temperature water bath and a pulling device at a precalibrated rate. The central rod of the machine is threaded and through a gear system provides movement to one end where the clip is fixed during initial placement. The other clip end is hooked at the fixed end of the machine. Two clips are thus pulled apart horizontally at a uniform speed of 50 ± 2.5 mm per minute. The machine may have provision to fix two or more moulds so as to test these specimens simultaneously.

Procedure

The bitumen sample is melted to a temperature of 75 to 100°C above the approximate softening point until it is fluid. It is strained through IS sieve 30, poured in the mould assembly and placed on a brass plate, after a solution of glycerine and dextrine is applied at all surfaces of the mould exposed to bitumen. Thirty to forty minutes after the sample is poured into the moulds, the plate assembly alongwith the sample is placed in water bath maintained at 27°C for 30 minutes. The sample and mould assembly are removed from water bath and excess bitumen material is cut off by levelling the surface using hot knife. After trimming the specimen, the mould assembly containing sample is replaced in water bath maintained at 27°C for 85 to 95 minutes. The sides of the mould are now removed and the clips are carefully hooked on the machine without causing any initial strain. Two or more specimens may be prepared in the moulds and clipped to the machine so as to conduct these tests simultaneously.

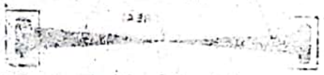
The pointer is set to read zero. The machine is started and the two clips are thus pulled apart horizontally. While the test is in operation, it is checked whether the sample is immersed in water at depth of at least 10 mm. The distance at which the bitumen thread of each specimen breaks, is recorded (in cm) to report as ductility value.

Results

The distance stretched by the moving end of the specimen upto the point of breaking of thread measured in centimeters is recorded as ductility value. It is recommended by ISI that test results should not differ from mean value by more than the following :

Repeatability : 5 percent

Reproducibility : 10 percent



Discussion

The ductility value gets seriously affected if any of the following factors are varied :

(i) pouring temperature
(ii) dimensions of briquette
(iii) improper level of briquette placement
(iv) test temperature
(v) rate of pulling.

Increase in minimum cross section of 10 sq. mm and increase in test temperature would record increased ductility value.

Applications of Ductility Test

A certain minimum ductility is necessary for a bitumen binder. This is because of the temperature changes in the bituminous mixes and the repeated deformations that occur in flexible pavements due to the traffic loads. If the bitumen has low ductility value, the bituminous pavement may crack, especially in cold weather. The ductility values of bitumen vary from 5 to over 100. Several agencies have specified the minimum ductility values for various types of bituminous pavement. Often a minimum ductility value of 50 cm is specified for bituminous construction.

The minimum ductility values specified by the Indian Standards Institution for various grades of bitumen available in India are given below :

Source of paving bitumen and penetration grade		Minimum ductility value, cm
Assam petroleum	A 25	5
	A 35	10
	A 45	12
	A 65, A 90 & A 200	15
Bitumen from sources other than Assam Petroleum	S 35	50
	S 45, S 65 & S 90	75

REFERENCES

1. Indian Standard Methods for Testing Tar and Bitumen; Determination of Ductility, IS : 1203, *Indian Standards Institution.*
2. Bituminous Road Construction, *Burmah Shell.*
3. Bituminous Materials in Road Construction, *D.S.I.R.*, H.M.S.O., London.
4. Indian Standard Specification for Paving Bitumen, IS : 73—1961, *Indian Standards Institution.*

PROBLEMS

1. Explain ductility of Bitumen and its significance.
2. How is ductility value expressed ?
3. Outline the ductility test procedure.
4. What is the minimum area of cross section of the ductility specimen ?
5. What are the precautions to be taken while finding the ductility value ?
6. What are the factors affecting the ductility test results ?

OBSERVATION SHEET

DUCTILITY TEST

(i) Grade of bitumen : =
(ii) Pouring temperature, °C =
(iii) Test temperature, °C =
(iv) Periods of cooling, minutes
(a) in air =
(b) in water bath before trimming =
(c) in water bath after trimming =

Test Property	Briquette Number (i)	(ii)	(iii)	Mean value
1. Ductility value (cm)				
2. Repeatability percent				
3. Reproducibility percent				

Remarks

19

Softening Point Test

INTRODUCTION

Bitumen does not suddenly change from solid-to-liquid state, but as the temperature increases, it gradually becomes softer until it flows readily. All semi-solid state bitumen grades need sufficient fluidity before they are used for application with the aggregate mix. For this purpose bitumen is sometimes cut back with a solvent like kerosene. The common procedure however is to liquify the bitumen by heating. The *softening point* is the temperature at which the substance attains particular degree of softening under specified condition of test. For bitumen, it is usually determined by *Ring and Ball* test. A brass ring containing the test sample of bitumen is suspended in liquid like water or glycerine at a given temperature. A steel ball is placed upon the bitumen and liquid medium is then heated at a specified rate. The temperature at which the softened bitumen touches the metal plate placed at a specified distance below the ring is recorded as the softening point of a particular bitumen. The apparatus and test procedure are standardized by ISI. It is obvious that harder grade bitumens possess higher softening point than softer grade bitumens. The concept of determining the softening point by ring and ball apparatus is shown in Figure 19.1.

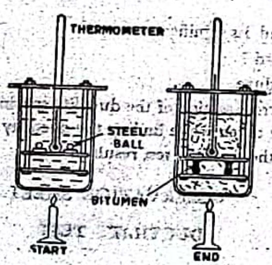


Figure 19.1 : Softening Point Test Concept

Apparatus

It consists of Ring and Ball apparatus.

(a) Steel Balls : They are two in number. Each has a diamer of 9.5 mm and weighs 2.5 ± .05 g.

(b) Brass Rings : There are two rings of the following dimensions.

Depth	6.4 mm	Inside diameter at top	17.5 mm
Inside diameter at bottom	15.9 mm	Outside diameter	20.6 mm

Brass rings are also placed with ball guides as shown in Figure 19.2.

(c) Support : The metallic support is used for placing pair of rings.

The upper surface of the rings is adjusted to be 50 mm below the surface of water or liquid contained in the bath. A distance of 25 mm between the bottom of the rings and top surface of the bottom plate of support is provided. It has a housing for a suitable thermometer.

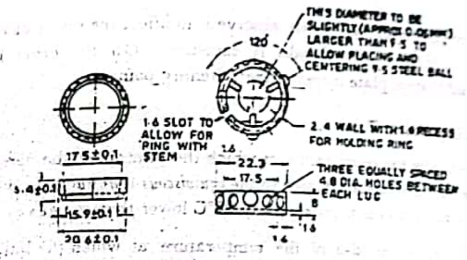


Figure 19.2 : Ring and Ball Guides

(d) Bath and stirrer : A heat reistant glass container of 85 mm diameter and 120 mm depth is used. Bath liquid is water for material having softening point below 80°C and glycerine for materials having softening point above 80°C. Mechanical stirrer is used for ensuring uniform heat distribution at all times throughout the bath.

Procedure

Sample material is heated to a temperature between 75 and 100°C above the approximate softening point until it is completely fluid and is poured in heated rings placed on metal plate. To avoid sticking of the bitumen to metal plate, coating is done to this with a solution of glycerine and dextrine. After cooling the rings in air for 30 minutes, the excess bitumen is trimmed and rings are placed in the support as discussed in item (c) above. At this time the temperature of distilled water is kept at 5°C. This temperature is maintained for 15 minutes after which the balls are placed in position. The temperature of water is raised at uniform rate of 5°C per minute with a controlled heating unit, until the bitumen softens and touches the bottom plate by sinking of balls. At least two observations are made. For material whose softening point is above 80°C, glycerine is used as a heating medium and the starting temperature is 35°C instead of 5°C.

Result

The temperature at the instant when each of the ball and sample touches the bottom plate of support is recorded as softening value. The mean of duplicate determinations is noted. It is essential that the mean value of the softening point (temperature) does not differ from individual observations by more than the following limits.

Softening point	*Repeatability*	*Reproducibility*
Below 30°C	2°C	4°C
30°C to 80°C	1°C	2°C
Above 80°C	2°C	4°C

Discussion

As in the other physical tests on bitumens, it is essential that the specifications discussed above are strictly observed. Particularly, any variation in the following point would affect the result considerably :

(i) quality and type of liquid
(ii) weight of balls
(iii) distance between bottom of ring and bottom base plate
(iv) rate of heating.

Impurity in water or glycerine has been observed to affect the result considerably. It is logical to observe lower softening point if the weight of balls is excessive. On the other hand, increased distance between bottom of ring and bottom plate increases the softening point.

Application of Softening Point Test

Softening point is essentially the temperature at which the bituminous binders have an equal viscosity. The softening point of a tar is therefore related to the *equiviscous temperature* (e. v. t.). The softening point found by the ring and ball apparatus is approximately 20°C lower than the e. v. t.

Softening point, thus gives an idea of the temperature at which the bituminous material attains a certain viscosity. Bitumen with higher softening point may be preferred in warmer place. Softening point is also sometimes used to specify hard bitumens and pitches.

The ranges of softening point specified by the Indian Standards Institution for various grades of bitumen are given below :

Bitumen Grades	Softening point, °C
* A 25 & A 35	55 to 70
* S 35	50 to 65
A 45, S 45 & A 65	45 to 60
S 65	40 to 55
A 90 & S 90	35 to 50
A 200 & S 200	30 to 45

* 'A' denotes bitumen from Assam Petroleum and 'S' denotes bitumen from sources other than from Assam Petroleum. Also see Table under 'Applications of penetration test'.

REFERENCES

1. Indian Standards Methods of Testing Tar and Bitumen : Determination of Softening Point, IS : 1205, *Indian Standards Institution*.
2. Bituminous Road Construction, *Burmah Shell*.
3. Bituminous Material in Road Construction, *D.S.I.R.*, H.M.S.O., London.
4. Indian Standard Specification for Paving Bitumen, IS : 73, 1961, *Indian Standards Institution*.

PROBLEMS

1. What is softening point ?
2. What does softening point of bituminous materials indicate ?
3. What are the applications of ring and ball test results ?
4. What are the factors which affect the ring and ball test results ?

OBSERVATION SHEET

SOFTENING POINT TEST

(i) Bitumen grade =
(ii) Approximate softening point =
(iii) Liquid used in the bath = water/glycerine
(iv) Period of air-cooling, minutes =
(v) Period of cooling in water bath, minutes =

Rate of heating :

Time, minutes	Temperature, °C	Time minutes	Temperature °C
0		11	
1		12	
2		13	
3		15	
4		16	
5		17	
6		18	
7		19	
8		20	
9			
10			

Observation :

Test Property	Sample no. 1		Sample no. 2		Mean value, Softening point
	Ball no.		Ball no.		
	(i)	(ii)	(i)	(ii)	
Temperature (°C) at which sample touches bottom plate					
Repeatability					
Reproducility					

Remarks :

20

Specific Gravity Test for Bitumen

INTRODUCTION

The density of a bitumen binder is a fundamental property frequently used as an aid in classifying the binders for use in paving jobs. In most applications, the bitumen is weighed, but finally in use with aggregate system, the bitumen content is converted on volume basis. Thus an accurate density value is required for conversion of weight to volume. The specific gravity is greatly influenced by the chemical composition of binder. Increased amounts of aromatic-type compounds cause an increase in the specific gravity. The Test procedure have been standardized by the ISI.

The specific gravity is defined by ISI as the ratio of the mass of a given volume of the bituminous material to the mass of an equal volume of water, the temperature of both being specified as 27°C±0.1°C.

Apparatus

There are two methods (i) Pyknometer method (ii) Balance method. For pyknometer method, the apparatus are specific gravity bottle of 50 ml capacity, ordinary capillary type with 6 mm diameter neck or wide mouthed capillary type bottle with 25 mm diameter neck can be used. For balance method, an analytical balance equipped with a pan straddle is used.

Procedure

Method 1 (Pyknometer method)

The specific gravity bottle is cleaned, dried and weighed alongwith the stopper. It is filled with fresh distilled water, stopper placed and the same is kept in water container for at least half an hour at temperature 27°C±0.1°C. The bottle is then removed and cleaned from outside. The specific gravity bottle containing distilled water is now weighed.

The bituminous material is heated to a pouring temperature and is poured in the above empty bottle taking all the precautions that it is clean and dry before filling sample materials. The material is filled upto the half taking care to prevent entry of air bubbles. To permit an escape of air bubbles, the sample bottle is allowed to stand for half an hour at suitable temperature cooled to 27°C and then weighed. The remaining space in the specific gavity bottle is filled with distill d water at 27°C, stopper placed and is placed in water container at 27°C. The bottle containing bituminous material and remaining water is removed, cleaned from outside and is again weighed.

Method 2. (Balance method)

In balance method the bitumen test specimen is cube shaped, about 12 mm on each edge. It is prepared by pouring the liquified bitumen sample in brass mould to provide the sample of required dimension and is cooled. The sample is weighed in air and is then weighed in distilled water maintained at 27°C ± 0.1°C to the nearest 0.1 mg.

Calculation

The specific gravity of the bituminous material is calculated as follows :

(i) Pyknometer method

$$\text{Specific gravity} = \frac{\text{weight of bitumonous material}}{\text{weight of equal volume of water at 27°C}} = \frac{(c-a)}{(b-a)-(d-c)}$$

a = weight of the specific gravity bottle

b = weight of the specific gravity bottle filled with distilled water

c = weight of the specific gravity bottle about half filled with bituminous material

d = weight of the specific gravity bottle about half filled with the material and the test with distilled water.

(ii) Balance Method

$$\text{Specific Gravity} = \frac{e}{(e-f)}$$

where

e = weight of the dry specimen

f = weight of the specimen when immersed in distilled water.

Discussion

It is necessary that all precautions are taken in making the specific gravity bottles thoroughly cleaned and dried in the first weighing. The surface of the specific gravity bottle should be cleaned dry after filling with water, before weighing. The test temperature should be firmly adhered to. Inaccurate balance would never give reproducible results. At least three measurements should be made for determining value of the specific gravity.

The specific gravity of all types of bituminous materials could be determined by the pyknometer method. However only the bitumen which is in semi-solid state at 27°C can be tested by the balance method. If the bituminous material is in liquid form at 27°C, the specific gravity may be found by the pyknometer method by completery filling the specific gravity bottle with the liquid material

Applications of Specific Gravity Test on Bituminous Materials

A knowledge of the correct specific gravity of bituminous materials have mainly two applications. First, to convert the specified bitumen content by weight to volume basis when the binder is measured by volume. Here it is neeessary to know the coefficient of expansion or the specfic gravity values at different temperatures.

Second, the specific firavity is useful to identifiy the source of a bituminous binder. Pure bitumen has a specific gravity in the range 0.97 to 1.02. (Higher penetration grade bitumen and cut back bitumen have lower range of specific gravity values). In case the bitumen contains mineral impurity, the specific gravity will be higher. Thus it is possible for a qualitative estimation of mineral impurity in bitumen. The specific gravity of tars depends on the type of carbonisation process used for their production. Vertical-retort tars have a specific gravity range 1.10 to 1.15 whereas horizontal-report and coke-oven tars have values in the range 1.18 to 1.25, (also please see Table 23.3 of Experiment No. 23).

The Indian Standards Institution specifies that the minimum specific gravity values of paving bitumen at 27°C shall be 0.99 for grades A 25, A 35, A 45, A 65, S 35, S 45, and S 65, 0.98 for A 90 and S 90 and 0.97 for A 200 and S 200. (For classifications of bitumen, see Table under Experiment No. 17, 'Applications of penetration test')

REFERENCES

1. Indian Standard Methods for Testing Tar and Bitumen : Determination of Specific Gravity, IS : 1202, *Indian Standard Institution.*
2. Bituminous Materials in Road Construction, *D.S.I.R.*, H.M.S.O., London.
3. Indian Standards Specification for Paving bitumen IS : 73-1961, *Indian Standards Institution.*

PROBLEMS

1. Explain the two methods of finding specific gravity of bituminous materials.
2. What precautions should be taken while finding the specific gravity ?
3. What are the applications of specific gravity test results ?

OBSERVATION SHEET

SPECIFIC GRAVITY TEST FOR BITUMEN

(i) Bitumen grade =

(ii) Test temperature =

A. Pyknometer Method

Sample No.	Weight of bottle, g	Weight of bottle + distilled water, g	Weight of bottle + half filled material, g	Weight of bottle + half filled material + distilled water, g	Specific gravity
	a	b	c	d	
1					
2					
3					
				Average value	

B. Balance Method

Sample No.	Weight of dry Sample	Weight of sample in distilled water, g	Specific gravity
	e	f	
1			
2.			
3			
		Average value	

Specific gravity at the testing temperature =

Remarks :

21

Viscosity Test

INTRODUCTION

Viscosity is defined as inverse of fluidity. Viscosity thus defines the fluid property of bituminous material. The degree of fluidity at the application temperature greatly influences the ability of bituminous material to spread, penetrate into the voids and also coat the aggregates and hence affects the strength characteristics of the resulting paving mixes. High or low fluidity at mixing and compaction has been observed to result in lower stability values. There is an optimum value of fluidity or viscosity for mixing and compacting for each aggregate gradation of the mix and bitumen grade. At high fluidity or low viscosity, the bituminous binder simply "lubricates" the aggregate particles instead of providing an uniform film thickness for binding action. Similarly, low fluidity or high viscosity does not enable the bitumen to coat the entire surface of aggregates in the mix easily and also resists the compactive effort and the resulting mix is hetrogeneous in character exhibiting low stability values. The ISI specifies a test procedure for liquid binders like cutback bitumen, emulsion and liquid tar. One of the methods by which viscosity is measured is by determining the time taken by 50 CC of the material to flow from a cup through specified orifice at a given temperature. This is illustrated in Figure 21.1. In the range of consistency of bituminous materials when neither orifice viscometer test nor penetration test could be conducted, *float test* may be carried out. Equipment like sliding plate microviscometer, and Brookfield viscometer are however in use

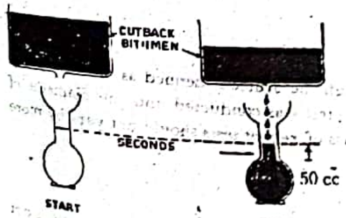


Figure 21.1 Viscosity Test Concept

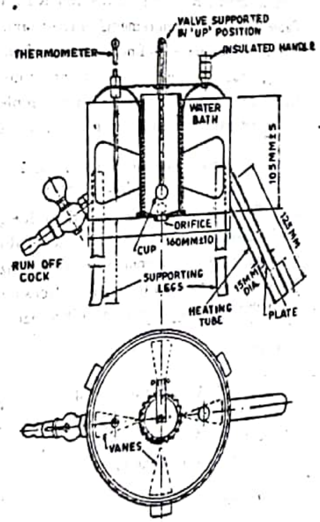


Figure 21.2 Tar Viscometer

for defining the viscous characteristics of the bitumen of all grades irrespective of testing temperature. The viscosity of bitumen can also be measured by capillary tube viscometer.

Apparatus

Ten millimetre orifice viscometer is specified for testing road tar and is called tar viscometer. 4.0 mm orifice is used to test cutback grades 0 and 1 and 10 mm orifice to test all other grades. Figure 21.2 shows the details of this apparatus. The apparatus consists of main parts like cup, valve, water bath, sleeves, stirrer, receiver and thermometers, etc.

Procedure

The tar cup is properly levelled and water in the bath is heated to the temperature specified for the test and is maintained throughout the test. Stirring is also continued. The sample material is heated at the temperature 20°C above the specified test temperature, and the material is allowed to cool. During this the material is continuously stirred. When material reaches slightly above test temperature, the same is poured in the tar cup, until the levelling peg on the valve rod is just immersed. In the graduated receiver (cylinder), 20 ml of mineral oil or one percent by weight solution of soft soap is poured. The receiver is placed under the orifice. When the sample material reaches the specified testing temperature within ± 0.1°C and is maintained for 5 minutes, the valve is opened. The stop watch is started, when cylinder records 25 ml. The time is recorded for flow upto a mark of 75 ml. (i.e., 50 ml of test sample to flow through the orifice).

The viscosity test on road tar is carried out using 10 mm orifice and the standard test temperature for road tar grades RT_1, RT_2, RT_3 and RT_4 are 35, 40, 45 and 55°C respectively. (See Table 21.1 and also Table 23.3 under requirements of road tar). In case the viscosity test is being carried out to classify a given sample of road tar or to find its grade, then the test should be first conducted at the lowest temperature of testing road tar, i.e., 35°C; if the time taken for 50 ml of the tar sample to flow through the 10 mm orifice is more than 55 secs. or if the sample does not flow freely, the test may be repeated at the next higher temperature, till the viscosity value falls in the specified range.

The viscosity test on cutback bitumen is carried out using 4.0 mm orifice for grades 0 and 1 (SC—0, MC—0, RC—0, SC—1, MC—1, RC—1), at 25°C. The tests for cutback grades 2 and 3 are carried out at 25°C using 10 mm orifice and those for grades 4 and 5 are carried out at 40°C using 10 mm orifice. For details of requirements of cutbacks, see Tables 23.1—a, b & c. If the viscosity of an unknown grade of cutbacks, is to be determined, the orifice size and the trial test temperature may be chosen using judgement. If the viscosity value of the trial test does not fall within the specified range, the test should be repeated by altering the test temperature or orifice size or both suitably.

Results

The time in seconds for 50 ml of the test sample to flow through the orifice is defined as the viscosity at a given test temperature. Therefore the temperature at which the test was conducted and the diameter of the orifice used should also be mentioned. The viscosity values of repeat tests should not vary by more than 4.0 percent from the mean value.

Discussion

The results of the viscosity test will get affected greatly if the test temperature of the sample is not correctly maintained through out the test. Erratic results are obtained due to clogging of the orifice and due to the presence of lumps in the sample of bituminous material.

Applications of viscosity test

Orifice viscosity test gives an indirect measure of viscosity of tars and cutbacks in seconds. Higher the duration, more viscous is the material.

The specified test temperatures and viscosity values of road tars of grades 1 to 5, using 10 mm orifice are given in Table 21.1. The specified orifice size, test temperature and viscosity values of different grades of cutback bitumen are given in Table 21.2. For further details of the properties and requirements of cutbacks and road tars, refer to Experiment No. 23.

The determination of viscosity by orifice viscometer in seconds is an indirect measure of viscosity. The absolute unit of viscosity is dyne-seconds per cm² or poise.

TABLE 21.1

Viscosity Values of Road Tars

Road tar grades	RT—1	RT—2	RT—3	RT—4	RT—5
Test temperature, °C	35	40	45	55	—
Viscosity range, secs.	30 to 55	30 to 55	35 to 60	40 to 60	—

TABLE 21.2

Viscosity Values of Cut back Bitumens

Cutback types SC, MC & RC of grades	0	1	2	3	4	5
Orifice size, mm	4.0	4.0	10	10	10	10
Test temperature, °C	25	25	25	25	40	40
Viscosity range, secs.	25 — 75	50 — 150	10 — 20	25—75	14—45	60 — 140

FLOAT TEST

There is a range of consistency of the bituminous materials for which neither an orifice viscometer test nor a penetration test could be used to define the consistency of the material. The consistency of the intermediate materials of this group is measured by the float test.

The apparatus consists of a float made of aluminium and a brass collar filled with the specimen material to be tested which is screwed to the float. The test specimen is filled in the collar (mould) cooled to a temperature of 5°C and screwed in to the float. The float assembly is floated in a water bath maintained at 50°C and the time required in seconds for water to force its way through the bitumen plug is noted as the float test value. The concept of float test is shown in Figure 21.3. The higher the float test value, the stiffer is the material.

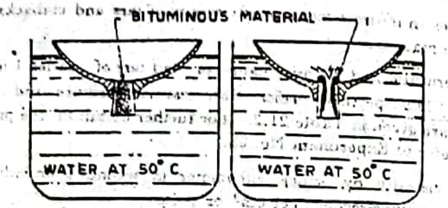


Figure 21.3. Float Test Concept

REFERENCES

1. Indian Standard Methods for Testing Tar and Bitumen : Determination of Viscosity, IS : 1206, *Indian Standards Institution.*
2. Bituminus Materials in Road Construction, *D.S.I.R.* H.M.S.O., London.
3. Highway Engineering, *S. K. Khanna* and *C.E.G. Justo,* Nem Chand & Bros., Roorkee.
4. Indian Standard Methods for Testing Tar and Bitumen : Float Test, IS : 1210, *Indian Standards Institution.*

PROBLEMS

1. Explain the term viscosity.
2. What are the different methods in determining the viscous characteristics of bituminous materials?
3. What is absolute unit for viscosity?
4. What are the uses of viscosity test?
5. Write a note on float test.
6. What are the precautions to be taken during viscosity test using orifice viscometer?
7. How is the grade of Tar/Cutback determined?

OBSERVATION SHEET

VISCOSITY TEST

(i) Material :
(ii) Grade :
(iii) Specified test temperature, °C =
(iv) Size of orifice, mm =
(v) Actual test temperature, °C =

Test property	Test run 1	2	3	Mean value
Viscosity in Seconds				
Repeatability, percent				

Remarks :

Flash and Fire Point Test

INTRODUCTION

Bituminous materials leave out volatiles at high temperatures depending upon their grade. These volatile vapours catch fire causing a flash. This condition is very hazardous and it is therefore essential to qualify this temperature for each bitumen grade, so that the paving engineers may restrict the mixing or application temperatures well within the limits. The flash point is the lowest temperature at which flash occurs due to the ignition of the volatile vapours when a small flame is brought in contact with the vapours of a bituminous product, gradually heated under standardised conditions. As mentioned above, this test gives an indication of the critical temperature at and above which suitable precautions should be taken to eleminate fire hazards during its application. When the bituminous material is further heated to a higher temperature, the material itself catches fire and continues to burn; the lowest temperature causing this condition is the fire point. The fire point is always higher than the flash point of a material. ISI vide specifications I.S : 1209—1958 gives the following definitions :

Flash point—"The flash point of a material is the lowest temperature at which the vapour of substance momentarily takes fire in the form of a flash under specified condition of test".

Fire point—"The fire point is the lowest temperature at which the material gets ignited and burns under specified condition of test".

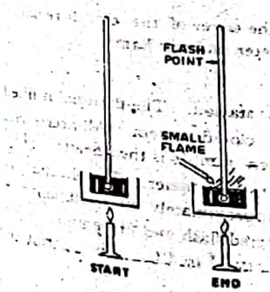
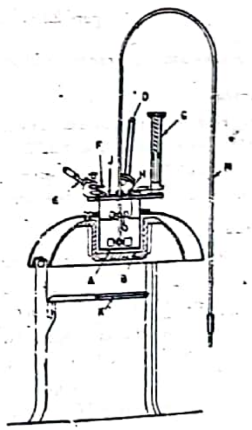


Figure 22.1 Flash Point Test Concept



A–Oil Cup
B–Heating Vessel
C–Stirrer
D–Thermometer
E–Ignition Burner
F–Pilot Burner
G–Spring handle
H–Revolving shutter
J–Orifice
K–Gauge Disc
L–Lifting Hooks
M–Optional form of stirrer operating device

Figure 22.2 Pensky-Martens Closed Tester

H—Minimum 23.9 mm Maximum 24.6 mm ; J—Minimum 13.5 mm Maximum 14.2 mm
K—Minimum 12.7 mm Maximum 13.7 mm ; N—Minimum 4.8 mm Maximum 5.6 mm
S—Approximately 19.0 mm ; U—Approximately 12.7 mm
Angles *p* are equal : Angle *r* — minimum 135° maximum 140°
Angle *t* — minimum 50° maximum 60° Angle *y* — minimum 10° maximum 15°

Figure 22.3. Cover for Pensky-Martens Closed Tester

The flash point test concept is illustrated in Figure 22.1.

Apparatus

(i) Pensky-Martens Closed Tester consist of cup, lid, stirring device, cover, shutter, flame exposure device etc. See Figures 22.2 and 22.3.

(ii) Pensky-Marten open Tester as above with the modification, that the cover of the cup is replaced by a clip which encircles the upper rim of the cup and carries the thermometer and test flame.

Procedure

All parts of the cup are cleaned and dried thoroughly before the test is started. The material is filled in the cup upto a filling mark. The lid is placed to close the cup in a closed system. All accessories including thermometer of the specified range are suitably fixed. The bitumen sample is then heated. The test flame is lit and adjusted in such a way that the size of a beed is of 4 mm diameter. The heating is done at the rate of 5° to 6°C per minute. The stirring is done at a rate of approximately 60 revolutions per minute. The test flame is applied at intervals depending upon the expected flash and fire points. First application is made at least 17°C below the actual flash point and then at every 1° to 3°C. The stirring is discontinued during the application of the test flame.

Results

The flash point is taken as the temperature read on the thermometer at the time of the flame application that causes a bright flash in the interior of the cup in closed system. For open cup it is the instance when flash appears first at any point on the surface of the material. The heating is continued until the volatiles ignite and the material continues to burn for 5 seconds. The temperature of the sample material when this occurs is recorded as the fire point.

Discussion

It is specified that in closed cup system, the test results should not differ from the mean by more than

5°C for materials flashing above 104°C and not more than 1°C from the mean for material flashing below 104°C. Sometimes bluish halo that surrounds the test flame is confused with true flash. For open cup system, it is specified by ISI that the mean value should not differ from the individual values by more than 8°C for flash point, and by 6°C for fire point.

Applications of Flash and Fire Point Tests

Different bituminous materials have quite different values of flash and fire points. When the bitumen or cutback is to be heated before mixing or application, utmost care is taken to see that heating is limited to a temperature well below the flash point. This is essential from safety point of view.

The minimum value of Flash point by Pensky Martens' Closed type apparatus specified by the ISI is 175°C for all the grades of bitumens (for both Assam petroleum and those from other sources).

The minimum specified flash point for rapid curing cutback bitumen of all grades is 26°C and that for medium curing type is 38° for grades 0 and 1 and 65° for grades 2 to 5. Slow curing cutbacks have minimum values ranging from 65 to 121°C as given in Table 23.

REFERENCES

1. Indian Standard Methods of Testing Tar and Bitumen : Determination of Flash Point and Fire Point, IS : 1209, *Indian Standards Institution*.
2. Bituminous Road Construction, *Burmash Shell*.
3. Indian Standards Specification for Paving Bitumen, IS : 73—1961, *Indian Standards Institution*.
4. Indian Standard Specification for Road Tar, IS : 215, ISI.
5. Indian Standard Specification for Cutback bitumen, IS : 217, ISI.

PROBLEMS

1. Define flash and fire points.
2. Briefly outline the flash point test procedure.
3. What is the significance of flash point test. Differentiate between flash point and fire point.

OBSERVATION SHEET

FLASH AND FIRE POINT TESTS

(i) Bitumen grade/Cutback type & grade : (ii) Type of equipment : Closed cup/Open cup

Rate of heating

Time, minutes	1	2	3	4	5	6	7	8	9	10	11	12
Temperature, °C												

Test property	Test number 1	2	3	Mean value
1. Flash point				
2. Fire point				
3. Variations from mean value				

Remarks :

23

Tests on Cutback, Emulsion and Tar

CUTBACK

Cutback bitumen is defined as the bitumen, the viscosity of which has been reduced by a volatile diluent. For use in surface dressings, some types of bitumen macadam and soil-bitumen stabilization, it is necessary to have fluid binder which can be mixed at relatively low temperatures or at atmospheric temperature. Hence to decrease the viscosity of the bituminous binder at low temperatures, the binder is blended with a volatile solvent. After the cutback mix is used in construction work, the volatile gets evaporated and the binder develops the binding properties. The viscosity of the cutback and the rate at which it hardens on the road depend on the characteristics and quantity of the bitumen and volatile oil used as the diluent.

Cutback bitumens are available in three types, namely (i) Rapid Curing (RC) (ii) Medium Curing (MC) and (iii) Slow Curing (SC).

This classification is based on the rate of curing or hardening after the application. Rapid curing cutback bitumen, are classified by ISI, on the basis of initial viscosity into six grades with designations RC—0, RC—1, RC—2, RC—3, RC—4, and RC—5, in the increasing order of initial viscosity. RC—0 is a rapid curing product of sufficiently low initial viscosity to be sprayed at the normal air temperature without heating, where as RC—4 and RC—5 are products of high viscosity, which can not be easily mixed with fine aggregate or soil at low temperatures.

Medium Curing (MC) cutback bitumens are classified on the basis of initial viscosity in to six grades MC—0, MC—1, MC—2, MC—3, MC—4, and MC—5 in the increasing order of viscosity. Similarly the Slow Curing (SC) cutbacks are also classified into six grades with designations SC—0, SC—1, SC—2, SC—3, SC—4 and SC—5. RC, MC and SC cutbacks of a particular grade (say RC—2, MC—2 and SC—2) have equal initial viscosity (in this example, 10 to 20 secs at 25°C with 10 mm orifice.)

RC, MC and SC cutback bitumens of the various grades mentioned above should comply with the requirements specified in IS 217—1961, with regard to the properties such as viscosity at different test temperatures, flash point, distillation fractions, residue from distillation up to the specified temperature, and tests residue from distillation.

The following tests are generally carried out on cutback bitumen

(i) Viscosity (with standard tar viscometer using 4 mm or 10 mm orifice)

(ii) Flash point test (closed type)

(iii) Distillation test (to find fractions of distillate up to 190, 225, 260, 315 and 360°C)

(iv) Tests on residue from distillation upto 360°C :

(a) Penetration test in the case of RC and MC and Float test in the case of SC cutback.

(b) Ductility test.

(c) Solubility test (in carbon disulphide)

The requirements of the various grades of cutback bitumen as specified by ISI are given in Table 23.1, a, b & c.

TABLE 23.1—a REQUIREMENTS OF RAPID CURING (RC) CUTBACK BITUMEN

Sl. No.	Characteristic	Requirement for Grade					
		RC—0	RC—1	RC—2	RC—3	RC—4	RC—5
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
(i)	Viscosity, Standard Tar Viscometer, in Sec. at :						
	(a) 25°C, 4 mm orifice	25 to 75	—	—	—	—	—
	(b) 40°C, 4 mm orifice	—	50 to 150	—	—	—	—
	(c) 25°C, 10 mm orifice	—	—	10 to 20	25 to 75	—	—
	(d) 40°C, 10 mm orifice	—	—	—	—	14 to 45	60 to 140
(ii)	Flash point, Pensky Martens closed Type. °C, Minimum	—	—	26	26	26	26
(iii)	Distillation fractions, percent by volume of total distillate up to 360°C :						
	(a) Up to 190°C Minimum	15	10	—	—	—	—
	(b) Up to 225°C Minimum	55	50	40	25	8	—
	(c) Up to 260°C Minimum	75	70	65	55	40	25
	(d) Up to 315°C Minimum	90	88	87	83	80	70
(iv)	Residue from distillation up to 360°C percent by volume (by difference) Minimum	50	60	67	73	78	82
(v)	Tests on residue from distillation up to 360°C :						
	(a) Penetration at 25°C. 100 g, 5 sec. in 1/100	80 to 120	80 to 120	80 to 120	80 to 120	80 to 120	80 to 120
	(b) Ductility at 27°, in cm, Min.	100	100	100	100	100	100
	(c) Matter soluble in carbon disulphide, percent by weight, Min.	99.5	99.5	99.5	99.5	99.5	99.5

TABLE 23.1-b REQUIREMENTS OF MEDIUM CURING (MC) CUTBACK BITUMEN

Sl. No.	Characteristics	Requirement for Grades					
		MC—0	MC—1	MC—2	MC—3	MC—4	MC—5
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
(i)	Viscosity, Standard Tar Viscometer, in sec. at :						
	(a) 25°C, 4 mm orifice	25 to 75	—	—	—	—	—
	(b) 40°C, 4 mm orifice	—	50 to 150	—	—	—	—
	(c) 25°C, 10 mm orifice	—	—	10 to 20	25 to 75	—	—
	(d) 40°C, 10 mm orifice	—	—	—	—	14 to 45	60 to 140
(ii)	Flash point, *Pensky Martens closed Type, °C, Min	38	38	65	65	65	65
(iii)	Distillation fractions, percent by volume, of total distillate up to 360°C						
	(a) Up to 225°C, Max	25	20	10	5	—	—
	(b) Up to 260°C	40 to 70	25 to 65	15 to 55	5 to 40	Not more than 30	Not more than 20
	(c) Up to 315°C	75 to 93	70 to 90	60 to 87	55 to 85	40 to 80	20 to 75
(iv)	Residue from direction upto 360°C percent by volume (by difference), Min.	50	60	67	73	78	82
(v)	Tests on residue from distillation upto 360° C.						
	(a) Penetration at 25°C 100 g 5 sec. in 1/100 cm	120 to 300	120 to 300	120 to 300	120 to 300	120 to 300	120 to 300
	(b) Ductility at 27°C in cm, Min	100	100	100	100	100	100
	(c) Matter soluble in carbon disulphide, percent by weight, Min.	99.5	99.5	99.5	99.5	99.5	99.5

TABLE 23.1—c REQUIREMENTS OF SLOW CURING (SC) CUTBACK BITUMEN

Sl. No.	Characteristics	Requirement for Grades					
		SC—0	SC—1	SC—2	SC—3	SC—4	SC—5
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
(i)	Viscosity, Standard Tar Viscometer, in sec. at :						
	(a) 25°C, 4 mm orifice	25 to 75	—	—	—	—	—
	(b) 25°C, 4 mm orifice	—	50 to 150	—	—	—	—
	(c) 25°C. 10 mm orifice	—	—	10 to 20	25 to 75	—	—
	(d) 40°C, 10 mm orifice	—	—	—	—	14 to 45	60 to 140
(ii)	Water, percent by weight, Max	0.5	0.5	0.5	0.5	0.5	0.5
(iii)	Flash point Pensky Martens closed Type °C, Min	65	65	79	93	107	121
(iv)	Total distillate upto 360°C, percent by volume	15 to 40	10 to 30	5 to 25	2 to 15	Max. 10	Max. 5
(v)	Residue of 100 penetration at 25°C, percent, Min.	40	50	60	70	75	80
(vi)	Tests on residue from distillation upto 360°C						
	(a) Float at 50°C, in sec.	15 to 100	20 to 100	25 to 100	50 to 125	60 to 150	75 to 200
	(b) Ductility at 27°C in cm, Min.	100	100	100	100	100	100
	(c) Solubility in carbon disulphide, percent by weight, Min.	99.5	99.5	99.5	99.5	99.5	99.5

EMULSION

A bitumen emulsion is a liquid product in which a substantial amount of bitumen is suspended in a finely divided condition in an aqueous medium and stabilized by means of one or more suitable materials. An emulsion is a two phase system consisting of two immiscible liquids, the one being dispersed as finite globules in the other.

The asphaltic bitumen or refined tar is broken up into fine globules and kept in suspension in water. A small proportion of an emulsifier is used to facilitate the formations of dispersions and to keep the globules of dispersed binder in suspension. The function of this emulsifier added in small proportions, is to form a protective coating around the globules of binder resisting the coalescence of the globules. Emulsifier usually adopted are soaps, surface active agents and colloidal powders. Half to one percent emulsifier by weight of finished emulsion is usually taken while preparing normal road emulsions. The average diameter of globules of bitumen is about 2 microns.

Usually the bitumen grades which are emulsified for road construction work are those with penetration values between 100 and 350. Emulsion of tar and bitumen mixtures are also prepared, but their use is restricted. Two common methods followed for the preparation of emulsions are, the colloid mill method and the high-speed mixer method. The manufactured emulsions are stored in air-tight drums.

When the emulsion is applied to the road, it breaks down and the binder starts coating the aggregates, though the full binding power develops slowly as and when the water evaporates. The first sign of break down of emulsion is shown by the change in colour of the film from chocolate brown of the emulsion to black colour of the bituminous binder.

The main advantages of emulsions are that (a) they can be used, without heating for preparing mixes (b) they are particularly useful for patch repair works and can be used even when the surface is wet or it is raining.

Three types of emulsified bitumen are prepared (i) *Rapid Setting—Type RS*, is a setting emulsified bitumen used for penetration and surface treatment (ii) *Medium Setting—Type MS*, is used for plant mixes with coarse aggregate. (iii) *Slow Setting—Type SS* is used for fine aggregate mixes.

Bitumen emulsion shall be homogeneous and it should not show undispersed bitumen after thorough mixing within 90 days after manufacture. The general properties of bitumen emulsion for roads are tested by the following test:

(i) Viscosity test

The viscosity of emulsion should be low enough to be sprayed through jets or to coat the aggregates in simple mixing. Viscosity of the emulsion is measured by standard Saybolt Furol viscometer at 25°C, in seconds.

(ii) Water Content

It is necessary to know the percentage water in the emulsion to estimate the actual binder quantity. Dean and Stark method is adopted for determining the water content.

(iii) Settlement test

The emulsion may settle due to sedimentation when left standing undistrurbed. The difference between the average bitumen residue at top and bottom after the specified storage period is reported as the settlement.

(iv) Demulsibility test

The percentage weight of residue of the emulsion obtained after being mixed with specified quantity of calcium chloride solution and sieved through the specified wire cloth is known as demulsibility.

(v) Miscibility in water

Addition of distilled water to the emulsion at specified temperature may cause co-agulation of the bitumen. The percentages of bitumen residue in top, middle and bottom levels of the cylinder are calculated in the modified test to judge the extent of miscibility in water.

(vi) Cement mixing test

This test is carried out to assess the stability of the emulsion when the aggregate contains large proportion of fines.

(vii) Coating test

This test is performed to find whether there is appreciable separation of the bitumen from the water of the emulsion and whether the stone aggregates are uniformly coated with the emulsion.

(viii) Sieve test

The percentage of residue retained on the specified sieve gives the measure of sedimentation of the emulsion during storage.

(ix) Test for particle charge

Deposition of appreciable layer of bitumen on the positive plate of the battery circuit and relatively clean bitumen-free negative plate indicates a negative particle charge of emulsion.

The physical and chemical requirements of the three type of emulsions, RS, MS and SS are given in Table 23.2.

TABLE 23.2 REQUIREMENTS OF BITUMEN EMULSION

Sl. No.	Characteristic	Type of emulsion		
		RS	MS	SS
(i)	Viscosity (Saybolt furol), secs. at 25°C	20 to 100	20 to 100	20 to 100
(ii)	Water content, percent max.	45	45	45
(iii)	Settlement, 5 days, percent, max	3	3	3
(iv)	Demulsibility, percent min.			
	(a) 35 ml of 0.02 N calcium chloride	60	—	—
	(b) 50 ml of 0.01 N calcium chloride	—	30	—
(v)	Modified miscibility with water, max difference of bitumen content	—	—	4.5
(vi)	Cement mixing test, percent max.	—	—	2.0
(vii)	Sieve test, percent residue on 850 micron sieve max.	0.1	0.1	0.5
(viii)	Particle charge	Negative	Negative	Negative

ROAD TAR

Tar is the viscous liquid obtained when natural organic materials such as wood and coal are carbonised or distructively distilled in the absence of air. Based on the material from which tar is derived, it is referred to as "wood tar" or "coal tar", the latter has been more widely used for road work because it is superior than the former. The three stages for the production of road tar are

(i) Carbonization of coal to produce crude tar

(ii) Refining or distillation of crude tar, and

(iii) Blending of distillation residue with distillate oil fraction to give desired road tar.

ISI has classified road tars into five grade : RT—1, RT—2, RT—3, RT—4 and RT—5, based on their viscosity and other properties. RT—1 is used for surface painting under exceptionally cold weather as this has very low viscosity. RT—2 is recommended for standard surface painting under normal Indian climatic conditions. RT—3 may be used for surface painting, renewal coats and premimixing chips for top course and light carpets. RT—4 is generally used for premixing tar macadam base course. For grouting purposes RT—5 may be adopted.

The various tests that are carried out on road tar specimens are.

(i) Specific gravity test

(ii) Viscosity test on standard tar viscometer

(iii) Equiviscous temperature (EVT),

(iv) Softening point
(v) Softening point of residue
(vi) Water content
(vii) Distillation fractions on distillation upto 200°C, 200°C to 270°C, 270°C, 270°C to 300°C
(viii) Phenols, percent by volume
(ix) Naphthalene percent by weight and
(x) Matter insoluble in toluene, percent by weight. The requirements for the five grades of road tars Based on the above test results are given by ISI, vide IS : 215-1961. Some of the requirements of the road tar as specified by ISI are given in Table 23.3.

TABLE 23.3 REQUIREMENTS FOR ROAD TAR

Sl. No.	Characteristic	Limits for Grades				
		RT 1	RT 2	RT 3	RT 4	RT 5
(i)	Specific gravity at 27°/27°C	1.6 to 1.26	1.16 to 1.26	1.18 to 1.28	1.18 to 1.28	1.18 to 1.28
(ii)	Viscosity by standard tar viscometer (10 mm)					
	(a) temperature of test, °C	35	40	45	55	—
	(b) viscosity in seconds	30 to 55	30 to 55	35 to 60	40 to 60	—
(iii)	Equiviscous temperature (EVT), °C	32 to 36	37 to 41	43 to 46	53 to 57	63 to 67
(iv)	Softening point, °C	—	—	—	—	45 to 50
(v)	Softening point of residue, °C Max.	48	50	52	54	[illegible]
(vi)	Water content, percent by weight, Max.	0.5	0.5	0.5	0.5	0.5
(vii)	Distillation fractions, percent by weight (g per 100 g) distilling :					
	(a) upto 200°C, Max.	0.5	0.5	0.5	0.5	0.5
	(b) from 200° to 270°C	5 to 12	2 to 9	1 to 6	0.5 to 4	0 to 4
	(c) from 270° to 300°C	4 to 10	4 to 8	3 to 6	2 to 7	1 to 5
	total distillation (b+c), Max	16	14	12	10	7
(viii)	Phenols, percent by volume, Max.	2.0	2.0	2.0	2.0	1.0
(ix)	Naphthalene, percent by weight, Max.	4.0	3.5	3.0	2.5	2.0
(x)	Matter insoluble in toluene, percent by weight, Max.	22.0	22.0	24.0	24.0	24.0

REFERENCES

1. Bituminous Materials in Road Construction, *D.S.I.R.*, H.M.S.O., London.
2. Indian Standard Specification for Cutback Bitumen IS : 217, *Indian Standards Institution*.
3. Indian Standard Specifications for Road Tar, IS : 215, *Indian Standard Institution*.
4. Indian Standard Specification for Bitumen Emulsion for Road, IS : 317, *Indian Standard Institution*.

PROBLEMS

1. What is a cutback ?
2. What are the different types of cutback ?
3. What are the special uses of cutbacks ?
4. What are the tests required to standardise a cutback ?
5. What is a bituminous emulsion ?
6. What is the composition of an emulsion ?
7. What are the special uses of bituminous emulsion ?
8. What tests are carried out on emulsion ?
9. What are the laboratory tests carried out on tar ?
10. What are the essential differences between tar and bitumen ?
11. What are the different grades of tars ? How are they classified ?

24

Joint Filler and Sealer

INTRODUCTION

Joints are required in cement concrete pavements of roads and runways to relieve stresses developed in the slabs by temperature and moisture variation. The gap provided between the slabs at expansion joints is about 2.0 to 2.5 cm wide and therefore these joints should be filled using suitable material to prevent the entry of water and grit and to provide even riding surface. The material used for filling should be resilient and non-extruding type; in other words the filler material should permit expansion and contraction of the concrete slab and should not extrude much while being compressed. The material used for filling the expansion joint gap is called joint filler. The common filler materials used are, bitumen impregnated preformed fibre, non-extruding cork, self expanding cork, cork-rubber, sponge rubber etc.

To make the joint effective it is also necessary to prevent the ingress of water and grit down the joint. This is achieved by using a sealing compound over the joint filler. Usually bituminous sealing compounds are used. The important requirements of sealing compounds are that they are not unduly affected by temperature changes and they resist the tendency to flow out of the joint under hot weather and do not loose resiliency during cold weather.

PREFORMED JOINT FILLERS

The tests and properties of bitumen-impregnated fibre fillers for expansion joints in concrete pavements are briefly presented here. Preformed strips of fibrous material are bonded together and uniformly impregnated with bitumen to form the fillers. The preformed strips of joint fillers should not get deformed or broken by handling or when exposed to atmospheric conditions. Specimens of size 10 × 10 cm are prepared for testing.

Recovery and Compression Tests

Recovery

The test specimen of size 10 × 10 cm and thickness of the filler strips is weighed and is centrally placed between the table of the compression testing machine and the flat metal plate. A dial gauge is placed through a U-shaped bridge within a hollow load transfer cylinder with the end of the dial gauge spindle touching the centre of the metal plate. The initial dial gauge reading of the specimen is noted accurate to 0.02 mm when subjected to the dead weight of the metal plate only. Load is applied at a rate of 1.0 mm per minute compression of the loading head till the test specimen is compressed to 50 percent of its thickness. Then the load is released immediately and the thickness of the specimen is noted 10 minutes after the release of the load and the recovery in thickness is found. The percentage recovery is calculated from :

$$\text{Recovery, \%} = \frac{100\, t_1}{t}$$

Where t_1 = thickness or specimen after the test, mm

t = original thickness of specimen before the test, mm

If the specimen shows a recovery of less than 70 percent of its original thickness the test is repeated, by subjecting the test specimen three applications of the load sufficient to compress it to 50 percent of its original thickness, and the load is released immediately, allowing the specimen to recover for 30 minutes

before the second and third application of the load. After the load is applied third time and released, the specimen is allowed to recover for one hour and the thickness is measured.

The ISI specifies that the specimen shall recover at least 70 percent of its original thickness.

Compression

The total maximum load required for the first application to compress the specimen by 50 percent is divided by the area of the metal plate (11 × 11 cm) and is recorded as the unit pressure in kg/cm². This pressure should be between 7 and 53 kg/cm² as per the ISI.

Loss of bitumen

After removing the bitumen squeezed out, the specimen is weighed and the loss in weight is calculated as a percentage of original weight of the specimen. The percent loss in weight should not exceed 3.0 percent.

Extrusion

The test specimen of size 10 × 10 cm is placed in a metal mould of same internal dimensions and three sides of the mould extending atleast 1 cm above the height of the specimen. The fourth side is such that the specimen can move freely on this edge only, when being compressed. A metal plate 10 × 10 × 1.2 cm size is placed, over the specimen in the mould and the assembly is placed over the testing machine as in the recovery test. The load is applied to compress the specimen to 50 percent thickness and the extrusion of the free edge of the specimen is measured accurate to 0.02 mm by a dial gauge or other suitable device. According to IS specifications, the extrusion of the free edge of the specimen should not exceed 6.5 mm.

Weathering

Accelerated weathering test is carried out by exposing two specimens first to 75°C for seven days, then immersing in water for 24 hours at room temperature. Then the specimens are placed on edge in a container immersed in water up to half the depth and are placed in a freezing chamber at −5°C till the water in the container freezes and then the container is placed in a water bath between 18 and 30°C till the ice surrounding the specimen gets completely melted. Such freeze-thaw cycles are repeated ten times. the specimen is removed from water and is placed in room temperature for 48 hours. The test specimen is examined for signs of disintegration. A good filler specimen should show no disintegration or separation of fibre.

Bitumen extraction and penetration of recovered bitumen

The bitumen is extracted from a weighed sample of filler specimen using benzene in a centrifugal extractor, the solution is distilled as specified and the penetration value of the recovered bitumen is determined. The penetration value should be between 25 and 100. The bitumen content in the specimen is also calculated.

SEALING COMPOUNDS

Sealing compounds are used for sealing the expansion joints above the filler material and also for filling contraction and longitudinal joints. The joint sealing compounds should form resilient and adhesive barrier in concrete joints and be capable of resisting the infiltration of water and the ingress of solid grit particles. The sealing compounds should also be able to stand the temperature variation and resist the tendency to flow out of joint or be picked up by vehicle tyres during hot weather and resist to become brittle or suffer loss of resiliency during cold weather. On heating, the sealing compound should acquire pouring consistency to enable pouring into the joint space. There are two grades of sealing compounds.

(i) Grade A, Normal and

(ii) Grade B, Jet Fuel Resistant

Grade B is used as joint sealing compound in runway construction for resisting the effects of jet aircraft and spillage of fuel. Tests for assessing the properties and requirements of joint sealing compounds are briefly given here.

(i) Pour Point test

The lowest temperature at which a joint sealing compound can be poured readily and uniformly is known as the pour-point. The pour point should not exceed 180°C.

(ii) Flow test

This test is used to assess the resistance of the sealing compound to flow in hot weather. The sealing compound is poured in three moulds of specified dimensions with a slot at the bottom. The excess material is removed and the moulds are weighed. They are then placed in an oven maintained at 45°C for five hours. The sealing compound which has flowed out is removed and the moulds with remaining compound are weighed again. The weight of compound flowed out is expressed as a percentage of original weight of the compound and the mean value should not exceed 6 percent.

(iii) Extension test

This test is used to assess whether the sealing compound can extend without fracture at 0°C. Cement mortar blocks are prepared and the sealing compound is poured into the joint space (see Figure 24.1). After cooling at room temperature for 16 to 24 hours, the specimen is stored at 0°C for a further period of 16 to 25 hours and then extended in a machine at a rate of 3 mm per hour for 6 mm placing chopped ice over the specimen. The four exposed faces of the test specimen are examined to see if each face remains adhering both the test blocks and if any cavity is formed. The total opening of cavity if formed should not exceed 40 mm².

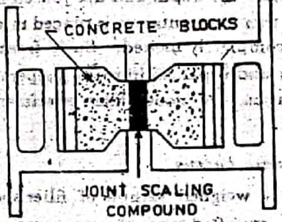


Fig. 24.1 Extension Test Set-up

(iv) Filler Settlement Test

Filler settlement apparatus consisting of five coaxial cylinders; they are filled with the sealing compound and placed in an oven at 20°C above the pour point for two hours. The cylinders are seperated and the soluble binder content of the joint sealing compound at the top, centre and bottom cylinders are determined and then the mineral filler in each are determined and expressed as a percentage of the total weight of the compound. The difference of the settlement value from the mean value should not exceed 10 percent.

(v) Test for Resistance to Grit

This test is meant to assess the resistance of joint sealing compound to the ingress of grit under the stress imposed by traffic. The sealing compound is poured into the container and is maintained at 35°C in a water bath for one hour. The impact plunger is placed in position and the dial gauge reading is taken. The weight is allowed to fall freely once each at 15 secs., 30 secs., 45 secs. and 5 minutes and the dial gauge reading is taken again. The test is repeated three times and the mean value of the penetration of the plunger between 0 and 5 minutes is expressed in units of 1/10 mm as the impact penetration value. This value should not exceed 10 for Grade A compound only.

(vi) Resistance to jet fuels

The sealing compounds of Grade B only are poured in containers and are kept under jet fuel at specified temperature for specified duration. The penetration value of the samples are determined and the increase in penetration value should not exceed 15. The change in weight after 7 days immersion in jet fuel (for Grade B only) should not exceed 1.0 percent. The requirements of joint sealing compound are given in Table 24.1.

TABLE 24.1 PHYSICAL REQUIREMENTS OF SEALING COMPOUNDS OF GRADES A and B

Sl. No.	Characteristics	Requirement
(i)	Pour point, Max.	180°C
(ii)	Softening point, Min.	75°C
(iii)	Increase in softening point after heating to 20°C above pour point for 3 hours, Max.	5°C
(iv)	Flow test, percentage, Max.	5
(v)	Extensibility, Min.	6 mm
(vi)	Filler settlement, difference from mean percent, Max.	10
(vii)	Resistance to grit penetration on impact test at 35°C in tenths of a mm, (for Grade A only), Max.	20
(viii)	Flash point, Min.	200°C
(ix)	Penetration, 25°C, Min.	15
	100 g, 5 s, 1/100, Max.	50
(x)	Jet fuel resistance :	
	(a) increase in penetration as measured in (ix) after 7 days immersion in jet fuel, Max (for Grade B only)	15
	(b) Change in weight, after 7 days immersion in jet fuel, percent, Max.	1.0

REFERENCES

1. Indian Standard Specification for preformed Fillers for Expansion Joint in Concrete Non-extruding and Resilient Type (Bitumen impregnated fiblre) IS : 1838, *Indian Standards Institution.*
2. Indian Standard Specification for Hot Applied Sealing Compounds for Joints in Concrete, IS : 1834, *Indian Standards Institution.*

PROBLEMS

1. What are the objects of joint filling and sealing materials.
2. What are their requirements ?
3. Specify the tests for assessing the suitability of (a) Joint fillers (b) Joint sealing compound.

Introduction

Bituminous mixes are used in the surface course of road and airfield pavements; some types of bituminous mixes are also used in base and/or binder course of flexible pavements. The bituminous mixes may be divided into two catagories based on the gradation of coarse and fine aggregates in the mix : (i) dense graded and (ii) open graded.

An example of dense graded bituminous mix is bituminous concrete, which is one of the highest quality bituminous component layers used in pavement surfacing. A suitably designed and constructed bituminous concrete surface course on a well designed and constructed pavement, should be able to withstand heavy traffic loads under adverse climatic conditions and also fulfil the requirements of structural and pavement surface characteristics.

The desirable bituminous mix properties include (i) stability (ii) density (iii) durability (iv) flexibility (v) resistance to skidding and (vi) workability during construction. Stability is defined as resistance of the paving mix to deformation under load and is thus a stress level which causes strain depending upon anticipated field conditions. Stability is a function of friction and cohesion. Fricional resistance is a function of both interparticle friction and friction imparted by bituminous materials. Cohesion is mainly offered by the factors that influence the mass viscosity of bitumen binder. Density is directly related to voids in the compacted mixture. Stability and density in general are related terms. If voids are restricted, the resulting strength properties of the paving mixes improve. Minimum voids requirement qualified for a given mix should be selected which would provide space for necessary densitification that may take place under traffic movement and expansion of bitumen at high temperatures. In the absence of this, the bitumen bleeds over the surface and causes skidding.

Durability is defined as the resistance of the mix against weathering which causes hardening and this depends upon loss of volatiles and oxidation. Tensile strain is introduced in the surfacing course consisting of bituminous mix when wheel loads ply over it. Excessive strain causes either cracking or rutting. Flexibility is a property of the mix that measures the extent of deformation before failure. Thus suitability of the given bituminous paving mix needs that all the above listed factors are considered.

Marshall stability test and Hveem stabilometer tests are largely used for the routine testing by many organisations in India and abroad. Criteria of the suitable mix design have been specified by Asphalt Institute based on the various factors discussed above. Thus all the above properties are indirectly accounted for in these tests. Marshall stability test adopted by U.S. Corps of Engineers is a semi-confined compression test which is comparable to a triaxial test with a cell pressure of about 0.7 kg/cm². Hveem Sabilometer embodies main characteristics of triaxial test.

Here in this part, two tests namely (i) Marshall Stability (ii) Hveem Stabilometer and Cohesiometer have been included. In pressenting the particulars of the tests, some deviations have been indicated to suit availability of equipment and prevalent climatic conditions. For Hveem Stabilometer kneading compactor is specified for use by Asphalt Institute; authors here have recommended to use Marshall compaction due to non-availability of the said equipment in India. This deviation thus may not fit in the exact definition of the test and this point should be kept in mind.

Marshall Stability Test

INTRODUCTION

Bruce Marshall, formerly Bituminous Engineer with Mississippi State Highway Department, USA formulated Marshall method for designing bituminous mixes. Marshall's test procedure was later modified and improved upon by U.S. Corps of Engineers through their extensive research and corelation studies. ASTM vide designation D 1559-62 T has standardised the test procedure. Generally, this stability test is applicable to hot-mix design using bitumen and aggregates with maximum size of 25 mm.

In this method, the resistance to plastic deformation of cylindrical specimen of bituminous mixture is measured when the same is loaded at the periphery at 5 cm per minute. This test procedure is used in designing and evaluating bituminous paving mixes. The test procedure is extensively used in routine test programmes for the paving jobs. There are two major features of the Marshall method of designing mixes namely, (i) density-voids analysis (ii) stability-flow tests. The Marshall stability of the mix is defined as a maximum load carried by a compacted specimen at a standard test temperature at 60°C. The flow value is the deformation the Marshall test specimen undergoes during the loading upto the maximum load, in 0.25 mm units. In this test an attempt is made to obtain optimum binder content for the type of aggregate mix and traffic intensity. The proposed design steps for the design of bituminous mix are given below :

(i) Select grading to be used.

(ii) Select aggregates to be employed in the mix.

(iii) Determine the proportion of each aggregate required to produce the design grading.

(iv) Determine the specific gravity of the aggregate combination and of the asphalt cement.

(v) Make up trial specimens with varying asphalt contents.

(vi) Determine the specific gravity of each compacted specimen.

(vii) Make stability tests on the specimens.

(viii) Calculate the percentage of voids, VMA and the percent voids filled with bitumen in each specimen.

(ix) Select the optimum bitumen content from the data obtained.

(x) Check the values of Marshall Stability, Flow, Voids in total mix and Voids filled with Bitumen obtained at the optimum bitumen content, with the design requirements. The design may be repeated if necessary after altering the gradation so as to fulfil the design requirements.

Apparatus

(a) *Mould Assembly :* Cylindrical moulds of 10 cm diameter and 7.5 cm height are required. It further consists of a base plate and coller extension. They are designed to be interchangeable with either end of cylindrical mould.

(b) *Sample Extractor :* For extruding the compacted specimen from the mould, an extractor suitably fitted with a jack or compression machine.

(c) *Compaction Pedestal and Hammer :* It consists of a wooden block capped with M.S. plate to hold the mould assembly in position during compaction. The compaction hammer consists of a flat circular tamping face 8.8 cm diameter and equipped with a 4.5 kg weight constructed to provide a free fall of 45.7 cm. Mould holder is provided consisting of spring tension device designed to hold compaction mould in place on the compaction pedestal.

(d) Breaking Head : It consists of upper and lower cylindrical segments or test heads having an inside radius of curvature of 5 cm. The lower segment is mounted on a base having two vertical guide rods which facilitate insertion in the holes of upper test head.

(e) Loading Machine : See Figure 25.1. The loading machine is provided with a gear system to lift the base in upward direction. On the upper end of the machine, a precalibrated proving ring of 5 tonne capacity is fixed. In between the base and the providing ring, the specimen contained in test head is placed. The loading machine produces a movement at the rate of 5 cm per minute. Machine is capable of reversing its movement downward also. This facilitates adequate space for placing test head system after one specimen has been tested.

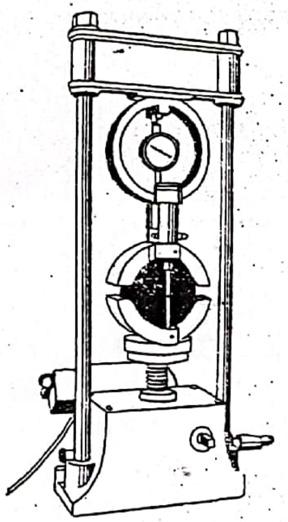


Figure 25.1 Marshall Stability Testing Machine

(f) Flow meter : One dial gauge fixed to the guide rods of a testing machine can serve the purpose. Least count of 0.025 mm is adequate. The flow value refers to the total vertical upward movement from the initial position at zero load to a value at maximum load. The dial gauge or the flow meter should be able to measure accurately the total vertical movement upward.

Besides the above equipment, the following are also required : (i) Ovens on hot plates, (ii) Mixing apparatus (iii) Water bath (iv) Thermometers of range upto 200°C with sensitivity of 2.5°C.

Procedure

In the Marshall method each compacted test specimen is subjected to the following tests and analysis in the order listed below :

(i) Bulk density determination

(ii) Stability and flow test

(iii) Density and voids analysis

Atleast three samples are prepared for each binder content and in all four to five binder contents are selected.

Preparation of Test Specimens

The coarse aggregates, fine aggregates and the filler material should be proportioned and mixed in such a way that final mix after blending has the gradation within the specified range. The specified gradation of mineral aggregates and bitumen binder as per IRC : 29 - 1968 are given in Table 25.1.

The aggregates and filler are mixed together in the desired proportion as per the design requirements and fulfilling the specified gradation. The required quantity of the mix is taken so as to produce a compacted bituminous mix specimen of thickness 63.5 mm, approximately.

Approximately 1200 g of aggregates and filler are taken and heated to a temperature of 175° to 190°C. The compaction mould assembly and rammer are cleaned and kept pre-heated to a temperature of 100°C to 145°C. The bitumen is heated to temperature of 121° to 138°C and the required quantity of first trial percentage of bitumen (say, 3.5% by weight of mineral aggregates) is added to the heated aggregate and thoroughly mixed using a mechanical mixer or by hand mixing with trowel. The mixing temperature for 80/100 grade bitumen may be around 154°C and that for 60/70 grade, about 160°C. The mix is placed in a mould and compacted by rammer, with 50 blows on either side. The compacting temperatures may be about 138°C for 80/100 grade bitumen and 149°C for 60/70 grade. The compacted specimen should have a thickness of 63.5 mm. The weight of the aggregate taken may be suitably altered to obtain a thickness of 63.5 ± 3.0 mm. Atleast two specimens, but preferably three or four specimens should be prepared at each trial bitumen content which may be varied at 0.5 percent increments upto about 7.5 or 8.0 percent.

The compactive effort is applied to each side of the specimen because laboratory compactive efforts should closely approach the maximum density that obtained in the pavement under traffic.

Tests

Specific Gravity of Compacted Specimens :

The specific gravity values of the different aggregates, filler and bitumen used are determined first. The theoretical specific gravity G_t of the mix is given by :

$$G_t = \frac{100}{W_1/G_1 + W_2/G_2 + W_3/G_3 + W_4/G_4}$$

Where W_1 = percent by weight of coarse aggregates

W_2 = percent by weight of fine aggregate

W_3 = percent by weight of filler

W_4 = percent by weight of bitumen in total mix.

G_1, G_2 and G_3 are apparent specific gravity values of the coarse aggregates, fine aggregates and filler respectively and G_4 is the specific gravity of bitumen.

Density and Voids Analysis

Soon after the compacted bituminous mix specimens have cooled to room temperature, the weight, average thickness and diameter of the specimen are noted. The specimens are also weighed in air and then in water. The bulk density value G_b of the specimen is calculated from the weight and volume. The voids analysis are made as given below :

$$V_v \% = \frac{100\,(G_t - G_b)}{G_t}$$

$$V_b \% = G_b \times \frac{W_4}{G_4}$$

$$VMA, \% = V_v + V_b$$

$$VFB, \% = \frac{100\ V_b}{VMA}$$

Where

V_v = air voids in the mix, %
V_b = volume of bitumen
VMA = voids in mineral aggregates, %
VFB = voids filled with bitumen, %

Marshall Stability and Flow Values

The specimens to be tested are kept immersed under water in a thermostatically controlled water bath maintained at 60° ± 1°C for 30 to 40 minutes. The specimens are taken out one by one, placed in the Marshall test head and the Marshall stability value (maximum load carried in kg. before failure) and the flow value (the deformation the specimen undergoes during loading upto the maximum load in 0.25 mm units) are noted. The corrected Marshall stability value of each specimen is determined by applying the appropriate correction factor, if the average height of the specimen is not exactly 63.5 mm. The correction factors are given in Table 25.2.

Determination of Optimum Bitumen Content

Five graphs are plotted with values of bitumen content against the values of :

(i) density G_b g/cm³
(ii) Marshall stability, S kg
(iii) voids in total mix, V_v %
(iv) flow value, F (0.25 mm units)
(v) voids filled with bitumen, VFB %.

Let the bitumen contents corresponding to maximum density be B_1, corresponding to maximum stability be B_2 and that corresponding to the specified void content V_v (4.0% in the case of dense AC mix) be B_3. Then the optimum bitumen content for mix design is given by :

$$B_o = (B_1 + B_2 + B_3)/3$$

The values of flow and VFB are found from the graphs, corresponding to bitumen content B_o. All the design values of Marshall stability, flow, voids and VFB are checked at the optimum bitumen content B_o, with the specified design requirements of the mix.

Design Requirements of the Mix

As per IRC : 29-1968, when the specimens are compacted with 50 blows on either face, the designed AC mix should fulfil the following requirements.

(i)	Marshall stability value kg (minimum)	= 340
(ii)	Marshall flow value, 0.25 mm units	= 8 to 16
(iii)	Voids in total mix, V_v %	= 3 to 5
(iv)	Voids in mineral aggregates filled with bitumen, VFB %	= 75 to 85

The highest possible Marshall stability values in the mix should be aimed at consistant with the other three requirements mentioned above. In case the mix designed does not fulfil any one or more of the design requirements, the gradation of the aggregates or filler content or bitumen content or combination of these are altered and the design tests are repeated till all the requirements are simultaneously fulfilled.

Job Mix Formula

The proportion in which the different aggregates, filler and bitumen are to be mixed are specified by weight or by volume for implementation during construction.

Calculations

The following values are either measured or computed :

(i) Bulk density
(ii) Stability
(iii) Flow
(iv) Percent air voids
(v) Percent voids filled with bitumen or tar
(vi) Percent voids in mineral aggregate.

Values (i), (ii) and (iii) are measured whereas values listed in (iv), (v) and (vi) are computed from the following :

Percent Air Voids

$$V_v = \frac{G_t - G_b}{G_t} \times 100$$

where, G_b = bulk density
G_t = theoretical specific gravity mixture

$$G_t = \frac{100}{\frac{W_1}{G_1} + \frac{W_2}{G_2} + \frac{W_3}{G_3} + \frac{W_4}{G_4}}$$

where, W_1 = percentage weight of coarse aggregate
W_2 = percentage weight of fine aggregate
W_3 = percentage weight of fines/Filler
W_4 = percentage weight of bitumen in total mix
G_1 = apparent specific gravity of coarse aggregates
G_2 = apparent specific gravity of fine aggregates
G_3 = apparent specific gravity of fines/Filler
G_4 = density of bitumen, g/cm³.

Percent of Voids in Mineral Aggregate (VMA)

$$VMA = V_v + V_b$$

where $V_b = G_b \times \frac{W_4}{G_4}$

Percent Voids Filled with Bitumen or Tar (VFB)

$$VFB = \frac{100\ V_b}{VMA}$$

Results

Above values obtained for four or five binder contents with a constant aggregate gradation are plotted on the graph for determining optimum binder content. A typical set of such plots is shown in Figure 25.2. From these plots, bitumen contents are determined corresponding to the following :

a. Maximum stability.
b. Maximum bulk density.

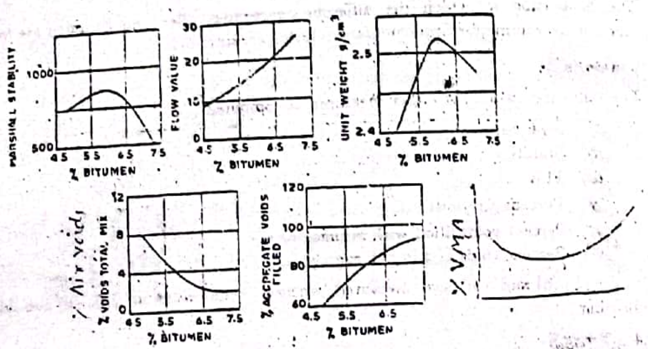


Figure 25.2 Bituminous Mix Design by Marshall Test

c. [redacted] depending upon the type of mix and the traffic intensity [redacted]

The optimum bitumen content of the mix is the numerical average of the three values for the bitumen contents determined as mentioned above.

Discussion

The Marshall stability test method is very simple and rapid method for designing bituminous mixes scientifically. The stability values obtained in this test procedure indirectly represent the strength of a paving mix at a zero vertical stress level which is critical.

Mixes with very high Marshall stability values and very low Flow values are not desirable as the pavements of such mixes may be brittle and are likely to crack under heavy traffic.

REFERENCES

1. Mix Design Methods for Asphalt Concrete and other Hot mix Types, *The Asphalt Institute*, U.S.A.
2. Design and Construction of Asphalt Pavements, *J.R. Martin* and *H.A. Wallance*, McGraw Hill Book Co.
3. Bituminous Materials in Road Construction, *D.S.I.R.*, H.M.S.O.
4. Asphalt Pavement Engineering, *H.A. Wallance* and *J.R. Martin*, McGraw Hill Book Co.
5. Tentative Specification for 4 cm Asphalt Concrete Surface Course, IRC : 29, 1968, *Indian Roads Congress.*

PROBLEMS

1. Why do we need to design bituminous mix ?
2. What are the essential properties of bituminous mixes ?
3. What is the significance of flow value in Marshall test ?
4. Why is the sample in Marshall test placed on its periphery while loading ?
5. What is the measure taken if a mix results in excessive voids ?
6. What is filler ?
7. What are different types of fillers ?
8. Does Portland cement, if used in bituminous mix improve strength ?
9. Briefly out line Marshall Stability test procedure ?
10. How is bituminous mix designed based on Marshall design approach ?

[redacted]

Sieve size	Percent passing, by weight	
	Grade 1	Grade 2
20 mm	—	100
12.5 mm	100	80—100
10 mm	80—100	70—90
4.75 mm	55—75	50—70
2.36 mm	35—50	35—50
600 micron	18—29	18—29
300 micron	13—23	13—23
150 micron	8—16	8—16
75 micron	4—10	4—10
[redacted]	[redacted]	[redacted]

Table 25.2 [redacted]

Volume of Specimen in Cubic Centimeters	[redacted]	Correction Factors
457—470	57.1	1.19
471—482	58.7	1.14
483—495	60.3	1.09
496—508	61.9	1.04
509—522	63.5	1.00
523—535	65.1	0.96
536—546	66.7	0.93
547—559	68.3	0.89
560—573	69.9	0.86

*Notes — (i) The measured stability of a specimen multiplied by the ratio for the thickness of the specimen is equal to the corrected stability for a 63.5 mm specimen.

(ii) Volume — thickness relationship is based on [redacted]

OBSERVATION SHEET

MARSHALL STABILITY TEST

Stability and flow value determinations

Type of grading of aggregate :

Grade of bitumen :

Mixing temperature, °C :

Compacting temperature, °C

Number of blows on either side =

Proving ring calibration factor =

Flow value dial, 1 division =

Sample No.	Bitumen content, percent	Maximum proving ring reading	Stability value, kg		Flow dial reading	Flow value, 0.25 mm. units
			Measured	Corrected		
1						
2	x_1 =					
3						
Average						
1						
2	x_2 =					
3						
Average						
1						
2	x_3 =					
3						
Average						
1						
2	x_4 =					
3						
Average						
1						
2	x_5 =					
3						
Average						

Density and void determinations

W_1 = G_1 = W_2 = G_2 =

W_3 = G_3 = W_4 = G_4 =

Sample No.	Bitumen content, percent	Height of sample, mm	Weight, g		Bulk Density G_b	V_v	V_b	VMA	VFB
			in air	in water					
1									
2	x_1 =								
3									
Average									
1									
2	x_2 =								
3									
Average									
1									
2	x_3 =								
3									
Average									
1									
2	x_4 =								
3									
Average									
1									
2	x_5 =								
3									
Average									

Plot the graphs as in Figure 25.2

(i) Maximum stability, kg = at bitumen content, % =

(ii) Maximum bulk density, g/cc = at bitumen content, % =

(iii) Percent air voids = at bitumen content, % =

Remarks

Average bitumen content =

26

Hveem Stabilometer and Cohesiometer Tests

INTRODUCTION

Franics N. Hveem, Materials and Research Engineer for the California Division of Highways advanced and developed the concept for designing paving mixtures based on stabilometer and cohesiometer tests. Extensive research and corelation studies have been reported in finalizing the test procedures. This method is used for the design of new construction or for evaluation of existing pavements. ASTM has standardized the test procedure by ASTM designation D-1560. The test method is used to design bituminous paving mix and for establishing the optimum content.

Three tests are carried out on the bituminous samples of 10 cm diameter and 6.35 cm height. The sequence of the test is as under :

(i) Swell test (ii) Stabilometer test (iii) Cohesiometer test.

The samples are recovered after completing test (i) and are subsequently used in test (ii) and test (iii).

Apparatus

The equipment required for testing the specimens are as under :

(a) Swell Test : The apparatus consists of a perforated plate, pan of water removable dial assembly and dial gauge.

(b) Hveem Stabilometer : Figure 26.1 shows the details of the equipment. Accessories include adjustable stage, mould assemble steel follower, and rubber bulb for introducing air into system.

The stabilometer consists of a cylindrical mould which can accommodate a specimen 10 cm diameter and 6.35 cm height resting over a rigid metal cylinder. The specimen is encased in the rubber membrane which acts as a inner well of the mould. Fluid pressure can be applied through the membrane, thus providing a lateral confinement to the specimen. The confining fluid pressure is applied by rotating a handle and is measured by a pressure gauge. Vertical pressure is applied through the loading head placed on the loading machine.

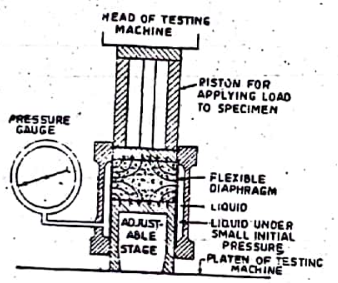


Figure 26.1 Stabilometer

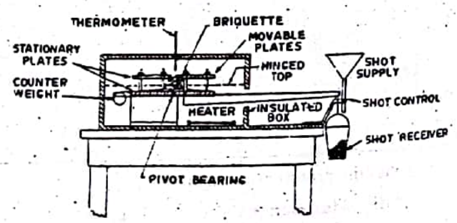


Figure 26.2 Cohesiometer

(c) Cohesiometer : Figure 26.2 shows a diagramatic sketch of equipment. It includes the system of insulated heating cabinet etc.

Procedure

The grading of materials and five bitumen contents, are taken as per experiment no. 25, Marshall Stability Test.

(a) Swell Test

The compacted specimen including mould is allowed to cool at room temperature for one hour. The perforated disc is placed over the specimen and the entire mould assembly fitted with tripod and dial gauge is placed in the water pan, 500 ml of water is allowed to stand over this disc. Initial reading is recorded and after one hour the final reading is also measured. The swell is measured nearest to 0.025 mm. The height of water lost during this process is recorded as permeability or the amount of water in mm that percolates through the test specimen.

(b) Stabilometer Test

The specimens recovered from swell test are placed in water bath or oven for a minimum period of one hour. The testing of the spcimen in stabilometer is done at a loading rate of 1.25 mm per minute. Before inserting the specimen in stabilometer, the apparatus is checked for its displacement value with metal dummy specimen. Bottom base of stabilometer is adjusted to provide rubber diaphram clearance of 6.35 cm to receive a specimen of this size. The specimen is then removed from the water bath/oven at 60°C and inserted in the stabilometer. The metallic follower is placed above the specimen and entire equipment is positioned in the compression machine adjustd at the rate of loading of 1.25 mm per minute.

To start with the operations on the stabilometer, the displacement pump handle is rotated to raise the fluid pressure recorded on dial gauge as 0.35 kg/cm^2. The displacement pump valve is promptly closed so as not to disturb the initial pressure of 0.35 kg/cm^2. Test loads are applied in sequence of 227 kg, 454 kg and each 454 kg upto a maximum of 2722 kg. Immediately after recording the reading on dial gauge at 2722 kg, the total load is reduced to 454 kg. Displacement pump valve is opened and the dial gauge reading is adjusted to 0.35 kg/cm^2. At this stage dial gauge on the displacement pump is adjusted to zero value and the handle is given turns rapidly at the rate of two turns per second until the pressure of 7 kg/cm^2 is recorded on test gauge. Exact number of turns are recorded to increase the test gauge reading from 0.35 kg/cm^2 to 7 kg/cm^2. The test load is now released to zero and the pressure on dial gauge is also brought to zero by rotating displacement handle in reverse direction.

(c) Bulk Density Measurement

It is done in the same manner as Marshall Stability Test

(d) Cohesiometer Test

The specimen which is recovered from testing on stabilometer and for bulk density determination is used for Cohesiometer test. Before commencing the test on cohesiometer the sample is placed in water bath or oven at 60°C for two hours. The cohesiometer device is calibrated to release lead shots into the bucket at the rate of 1800 $\pm$ 20 grams per minute. The sample is arranged in position and the cabinet temperature of the cohesiometer brought to 60 $\pm$ 1°C before commencement of the test. Released pin is pulled out and shots are allowed to flow until the specimen breaks. The shots received in the bucket are weighed accurately.

Stabilometer Value

The following equation is used in computing stability or stabilometer value :

$$S = \frac{22.2}{\frac{P_h D_2}{P_v - P_h} + 0.222}$$

where

S = relative stability

D_2 = displacement on specimen represented as number of turns of pump handle to raise P_h from 0.35 to 7 kg/cm²

P_v = vertical pressure at 28 kg/cm² or 2275 kg total load

P_h = horizontal pressure corresponding to P_v = 28 kg/cm²

Cohesiometer Value

The cohesiometer value is calculated using the following relationship :

$$C = \frac{L}{W(0.2\,H + 0.0176\,H^2)}$$

where

C = cohesiometer value

L = weight of shots, g

W = diameter or width of specimen, cm

H = height of specimen, cm

Density and voids analysis

Using the bulk density of the test specimens, percent air voids etc., are calculated as explained in Marshall Stability test.

Graphs are plotted for the different binder contents with the values of the following :

(i) Unit weight

(ii) Percent air voids

(iii) Hveem stability

(iv) Cohesiometer value.

Discussions and Application

Hveem method of design has been backed by extensive performance studies. Asphalt Institute recommends the following design criteria :

Value	Traffic		
	Light	Medium	Heavy
Stabilometer	30+	35+	37+
Cohesiometer	50+	50+	50+
Swell	less than 0.75 mm		

While making use of these requirements, optimum binder content for the design mix is taken as the highest percent of bitumen the mix accommodates without sacrificing void content i.e. 4 to 5 percent.

It is not essential to aim at highest stability value as mix performance studies have proved the suitability of the design criteria mentioned above. The cohesiometer value can be raised by modifying aggregate grading with higher filler content or using harder penetration grade bitumen.

REFERENCE

1. Mix Design Method for Asphalt Concrete, *The Asphalt Institute*, U.S.A.
2. Asphalt Pavement Engineering *H. A. Wallace* and *J. R. Martin*, McGraw Hill Book Co.

PROBLEMS

1. Outline Hveem Stability test.
2. How is Cohesiometer test carried out ?
3. Enumerate the mix design method based on Hveem Stability test.

OBSERVATION SHEET

(i) Type of aggregate =

(ii) Type of grading =

(iii) Bitumen type =

A. **Hveem Stabilometer Test**

Particulars*	A	B	C	D	E	Remarks
Bitumen content by weight of aggregate, percent						
Bitumen content by weight of mix, percent						
Weight in air, g						
Weight in water, g						
Bulk volume, cc						
Bulk density, g/cc						
Air voids, percent,						
Unit weight, g/cc						

*Average values of three samples

OBSERVATION SHEET

HVEEM STABILOMETER AND COHESIOMETER TESTS

(i) Type of aggregate =
(ii) Type of grading =
(iii) Bitumen content =
(iv) Bitumen grade =

STABILOMETER TEST

Total load kg	Pressure, kg/cm²	Binder content, percent					Remarks
		A =	B =	C =	D =	E =	
227	5.8						
454	5.6						
908	11.2						
1362	16.8						
1816	22.4						
2270	28.0						
2724	33.6						

COHESIOMETER TEST

Temperature, °C	
Weight of shots, g	
Cohesiometer value	

Remarks :

List of essential equipment

Equipment for highway material testing laboratory

PART I TESTS ON SUBGRADE SOILS AND STABILIZED SOILS

1. Set of I.S. sieves
2. Sieve Shaker
3. Liquid Limit Apparatus
4. Shrinkage Limit Apparatus
5. Compaction Test apparatus
   (a) For I.S. Light Compaction
   (b) For I.S. Heavy Compaction
6. C.B.R. Apparatus
7. Compression Testing Machine, Capacity 5 Tons, strain controlled at 1.25 mm/min.
8. Apparatus for Hydrometer and Pipette analyses
9. Triaxial apparatus
10. Plate Bearing Test Equipment
11. Iowa Bearing Value Apparatus
12. Extrusion Tests Apparatus for soil-bitumen mix
13. North Dakota Cone Test
14. Field Density Test-Sand Replacement Apparatus and core cutter

PART II TESTS ON AGGREGATES

1. Indian Standard Sieves for aggregates
2. Apparatus for determining Angularity Number
3. Riffle
4. Compression Testing Machine, Capacity 50 tonnes
5. Aggregate Crushing a Value Apparatus
6. British (Dorry) Abrasion Machine
7. Los Angeles Abrasion Machine
8. Deval Abrasion Testing Machine
9. Aggregate Impact Test Machine
10. Diamond Core Cutting, Sawing and Grinding Machines for preparation of Cylindrical Stone Specimens
11. Length and Thickness Gauges

PART III TESTS ON BITUMINOUS MATERIALS

1. Bitumen Sampler
2. Penetration Apparatus (Hand operated and automatic)
3. Ring and Ball Apparatus
4. Ductility Testing Machine
5. Tar viscometer
6. Pensky-Marten Closed Tester
7. Pensky-Marten Open Tester
8. Float Test Apparatus
9. Dean and Stark Apparatus

PART IV TEST ON BITUMINOUS MIXTURES

1. Marshall Stability Test Apparatus
2. Hveem Stabilometer and Cohesiometer
3. Triaxial Cell Assembly and Testing Machine
4. Hubbard Stability Test equipment

GENERAL EQUIPMENT

1. Thermostatically Controlled Drying Ovens
   (a) Controlled at 105°—110°C ± 1°C
   (b) Range 40°—250°C ± 1°C
2. Weighing Machines
   (a) Capacity 200g, Sensitivity 0.01g.
   (b) Capacity 1 kg, Sensitivity 0.1 g.
   (c) Capacity 5 kg, Sensitivity 0.5g.
   (d) Capacity 10 kg, Sensitivity 5g.
   (e) Capacity 50 kg, Sensitivity 10g.
3. Freezing Cabinet, Thermostatically Controlled, Range—23°C to 30°C.
4. Distillation apparatus, Flasks Condensers etc.
5. Water Bath, Thermostatically Controlled, Range 0 to 100°C±2°C
6. Dial gauges of least count 0.01 and 0.02 mm
7. Jacks, Sample Extractor, Moulds of different Sizes.
8. Other Measuring Equipment like Scales, Vernier
9. Burette, Pipette, Jars, Thermometers etc. Heating Devices and other Accessories
10. Reagents, Acids, Carbon Disulphide, Carbon Tetrachloride, Toluene, Mercury etc.

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