# Chapter 6
# Highway Materials

## 6.1 SUBGRADE SOIL

### 6.1.1 Significance of Subgrade Soil

Subgrade soil is an integral part of the road pavement structure as it provides the support to the pavement from beneath. The subgrade soil and its properties are important in the design of pavement structure. The main function of the subgrade is to give adequate support to the pavement and for this the subgrade should possess sufficient stability under adverse climate and loading conditions.

The formation of waves, corrugations, rutting and shoving in black top pavements and the phenomena of pumping, blowing and consequent cracking of cement concrete pavements are generally attributed due to the poor subgrade conditions.

When soil is used in embankment construction, in addition to stability incompressibility is also important as differential settlement may cause failures. Compacted soil and stabilized soil are often used in sub-base or base course of highway pavements. The soil is therefore considered as one of the principal highway materials.

### 6.1.2 Characteristics of Soil

Soil consists mainly of mineral matter formed by the disintegration of rocks, by the action of water, frost, temperature, pressure or by plant or animal life. Based on the individual grain size of soil particles, soils have been classified as gravel, sand, silt and clay. The characteristics of soil grains depend on the size, shape, surface texture, chemical composition and electrical surface charges. Moisture and dry density influence the engineering behaviour of a soil mass.

### 6.1.3 Desirable Properties

The desirable properties of soil as a highway material are

(i) Stability

(ii) Incompressibility

(iii) Permanency of strength





(iv) Minimum changes in volume and stability under adverse conditions of weather and ground water

(v) Good drainage, and

(vi) Ease of compaction.

The soil should possess adequate stability or resistance to permanent deformation under loads, and should possess resistance to weathering, thus retaining the desired subgrade support. Minimum variation in volume will ensure minimum variation in differential expansion and differential strength values. Good drainage is essential to avoid excessive moisture retention and to reduce the potential frost action. Ease of compaction ensures higher dry density and strength under particular type and amount of compaction.

### 6.1.4 Index Properties of Soil

The wide range of soil types available as highway construction materials have made it obligatory on the part of the highway engineer to identify and classify the different soils. The soil properties on which their identification and classification are based are known as *index properties*. The index properties which are usually used are *grain size distribution, liquid limit and plasticity index*. Further the properties which are some times used are *shrinkage limit, field moisture equivalent centrifuge moisture equivalent and compacted dry density*.

*Grain Size Analysis*

The grain size distribution is found by mechanical analysis. The components of soils which are coarse grained may be analysed by *sieve analysis* and the soil fines by *sedimentation analysis*. The grain size analysis or the mechanical analysis is hence carried out to determine the percentage of individual grain size present in a soil sample.

The sieve analysis is a simple test consisting of sieving a measured quantity of the material through successively smaller sieves; the weight retained on each sieve is expressed as a percentage of the total weight of sample. The sedimentation principle, that the larger grains in a suspension settle faster, is used for finding the grain size distribution of fine soil fraction passing 75 micron sieve. Two methods of test viz. : Hydrometer method and Pipette method are used based on sedimentation principle. The details of the grain size analysis tests as well as all tests on highway materials have been given by the authors in their book, *Highway Material Testing*.

*Consistency Limits and Indices*

The physical properties of fine grained soils, especially of clays differ very much at different water contents. A clay may be almost in a liquid state, or it may show plastic behaviour or may be stiff depending on the moisture content. Plasticity is a property of outstanding importance for clayey soils, which may be explained as ability to undergo changes of 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 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 very small shearing force. The liquid limit is usually determined in the laboratory using a mechanical device.

*Plastic limit* may be defined in general terms, as the minimum moisture content at which the soil remains in a plastic state. This 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 between the liquid and plastic limits. Plasticity index thus indicates the range of moisture content over which the soil is in 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 lowest water content that can occur in clayey soil sample which is completely saturated.

Consistency limits and the plasticity index vary for different soil types and therefore these properties are generally used in the identification and classification of soils. Generally soils having high values of liquid limit and plasticity index are poor as engineering materials. Both liquid limit and plastic limit depend on the type and amount of clay in soils. The plasticity index generally depends only on the amount of clay present; giving an indication of clay content in soil. In soil having same values of liquid limit, but with different values of plasticity index; it is generally found that rate of volume change and dry strength increases and permeability decreases with increase in plasticity index. In soils having same values of plasticity index but different values of liquid limit, it is seen that compressibility and permeability increase and dry strength decreases with increase in liquid limit. Thus the values of liquid limit and plasticity index help in classifying the cohesive soils.

In addition to the above tests certain other properties have also been some time used in identifying and classifying soils. These include shrinkage limit, field moisture equivalent, centrifuge moisture equivalent and compaction characteristics of the soils.

*Field moisture equivalent* of a soil is the moisture content at which the demands for absorbed water are fully satisfied. The *centrifuge moisture equivalent* of a soil is the moisture content retained against a force of 1000 times gravity for one hour. These tests are seldom carried out now-a-days. In most of the soil classifications systems that are commonly in use, the classifications are based on the grain size distribution (by sieve analysis), liquid limit and plasticity index of the soils.

### 6.1.5 Soil Classification Based on Grain Size

There are several classification systems for soil grains based on grain size of soil, according to which soils have been classified as gravel, sand, silt and clay. The exact limits of grain size for each of these components are not same in all these classifications. The most widely accepted grain size classification system is the *M.I.T. classification system*. The Indian Standards Institution (I.S.I.) has also adopted the same limits of M.I.T. system for the *Indian Standard Classification System* of soil grains. The limits of the grain size for each component as per this system are shown below :

| Gravel | Sand | | | Silt | | | Clay | | |
|---|---|---|---|---|---|---|---|---|---|
| | Coarse | Medium | Fine | Coarse | Medium | Fine | Coarse | Medium | Fine or Colloidal |
| 2.0* | 0.6 | 0.2 | 0.06 | 0.02 | 0.006 | 0.002 | 0.0006 | 0.0002 | |

*Values are in mm



*Textural Classification*

The textural classification system is based on grain size distribution of the soil and is helpful in classifying a soil which contains different soil components such as sand silt and clay. A typical textural classification chart for subgrade soils [illegible] sand and smaller grain sizes (suggested by *U. S. Bureau of Public Roads*) is given in Fig. 6.1

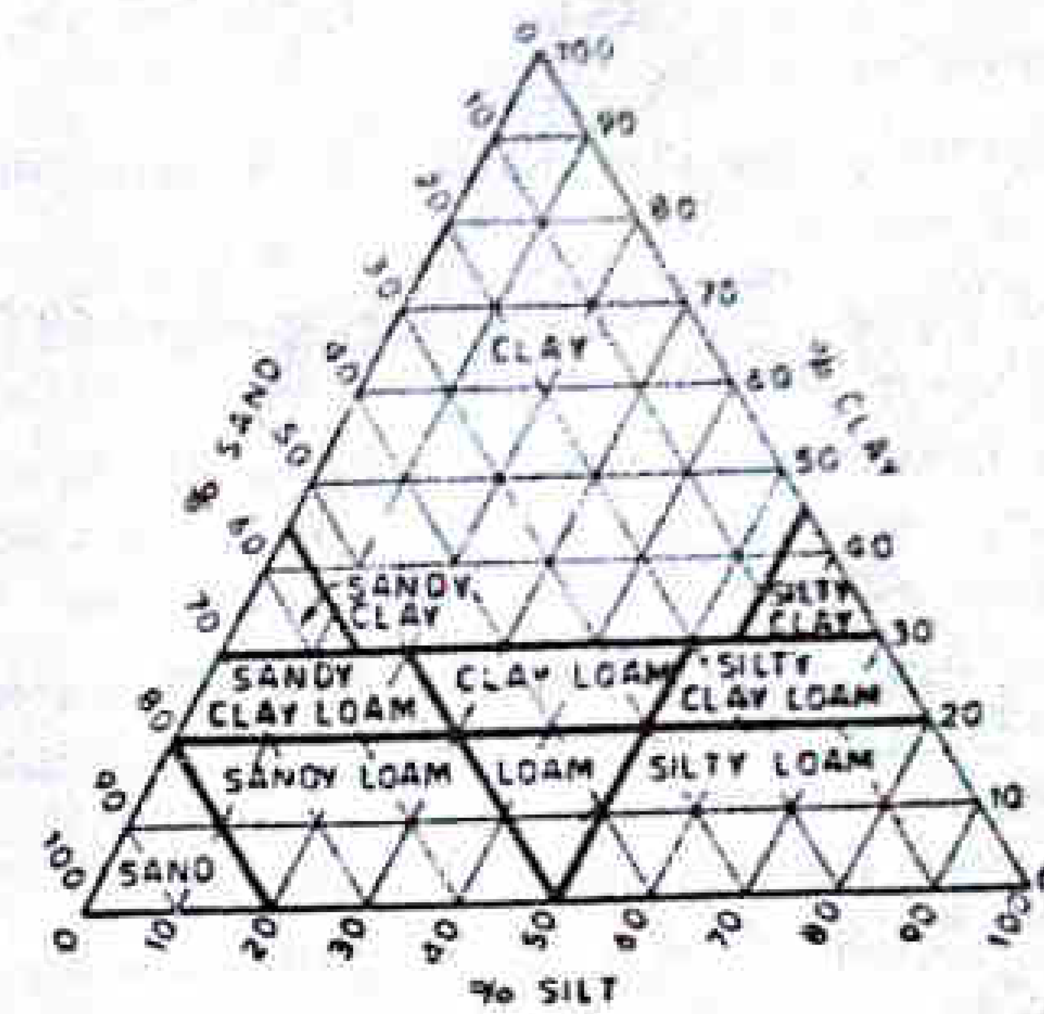



**Fig. 6.1 Textural Classification Chart**

### 6.1.6 Soil Classification Systems

The various soil classification systems in use in the field of highway engineering are :

(i) Burmister descriptive classification

(ii) Casagrande soil classification

(iii) Unified soil classification of Revised Casagrande soil classification and I. S. soil classification systems.

(iv) U. S. Public Roads Administration (PRA) classification

(v) Highway Research Board (HRB) or American Association of State Highway Officials (AASHO) classification or Revised PRA classification.

(vi) Federal Aviation Agency (FAA) classification.

(vii) Civil Aeronautic Administration (CAA) classification.

(viii) Compaction classification.

Of these systems, the Unified soil classification system has been very widely accepted in general for the classification of soils for civil engineering purposes and the H.R.B. classification or the revised PRA system is adopted for the classification of subgrade soils in Highway Engineering. Hence these two classification systems are given here :

**Unified soil classification system**

The Casagrande classification system which was developed in 1942 to classify soils, was later revised, modified and adopted by both the U. S. Crops of Engineers and the U. S. Bureau of Reclamation and was re-named as *Unified Soil Classification System*.

The soils are divided into two board groups, coarse grained and fine grained, based on grain size. The coarse grained soils include gravels (groups symbol G) and sand (group symbol S). Each of these component have been subdivided as well graded (symbol W), well graded with clay binder (symbol C), poorly graded (symbol P) and material containing considerable proportion of silt (symbol M). Thus a well graded gravel is GW, well graded sand SW, poorly graded gravel GP and so on.

The fine grained soil with more than half passing 200-mesh I. S. sieve (0.074 mm size) have been divide into two groups :

(i) Soils with liquid limit less than 50 percent or soils with low to medium compressibility (group symbol L).

(ii) Soils with liquid limit greater than 50 percent or soils with high compressibility (group symbol H).

Symbols M, C and O have been assigned to inorganic silts including very fine sand, inorganic clay and organic soils (silt and clay) respectively. Thus inorganic silt with low plasticity is ML, inorganic silt with high plasticity is MH and so on.

Unified soil classification groups, symbols and limits of test properties are given in Table 6.1. This soil classification system makes use of the results of sieve analysis, liquid limit and plastic limit tests. The classification group of the fine grained soil is found by making use of plasticity chart given in Fig. 6.2.

**Table 6.1 Unified Soil Classification**

| Major Division | | | Symbol | Brief description of soil types | Laboratory test results |
|---|---|---|---|---|---|
| Coarse grained soils, more than 50% material larger than No. 200 sieve size (0.074 mm) | Gravel, "G" more than half of coarse fraction larger than 4.76 mm sieve | Clean gravel with little or no fines* (less than 5%) | GW | Well graded gravels and gravels sand mixtures | Uniformity coefficient- $Cu = D_{60}/D_{10} > 6$ Gradation coefficient, $Cg = D^2_{30}/D_{60} D_{10} = 1$ to 3 |
| | | | GP | Poorly graded gravels and gravel -sand mixtures | Not meeting the Cu and Cg requirement of GW |
| | | Gravel with appreciable proportion of fines* (more than 12%) | GM | Silty gravel and gravel sand mixtures | |
| | | | GC | Clayey gravels and gravel-sand-silt mixtures | |
| | Sands, "S" more than half of coarse fraction smaller than 4.76 mm | Clean sand with little or no fines* (Less than 5%) | SW | Well graded sand and gravelly sands | Cu < 4 Cg = 1 to 3 |
| | | | SP | Poorly graded sands and gravelly sand | Not meeting the Cu and Cg requirements of SW |
| | | Sands with appreciable proportion of fines* (more than 12%) | SM | Silty sand and sand-silt mixtures | |
| | | | SC | Clayey sand and sand clay mixtures | |
| Fine grained soils, more than 50% materials smaller than No. 200 sieve size (0.074 mm ) | Silts and clays with liquid limit less than 50 – "L" | | ML | Inorganic silts, very fine rock floor, clayey silt or fine sand | Classification by plasticity chart (Fig. 6.2) |
| | | | CL | Inorganic clays, gravelly sandy or silty | |
| | | | OL | Organic silt and silty clays | |
| | Silts and clays with liquid limit greater than 50 – "H" | | MH | Inorganic silt elastic and Micaceous silts | |
| | | | CH | Inorganic fat clays | *Fines are those materials smaller than No. 200 sieve size or 0.074mm size |
| | | | OH | Organic silt and clays | |
| | Highly organic soils | | Pt | Peat and other highly organic soils | |

## I.S. soil classification

The Indian Standard Institute (ISI) has also adopted a soil classification system based on the Unified Soil Classification System. There is only slight variation in some of the subgroups and their symbols. The particulars of IS soil classification is given in Table 6.2.



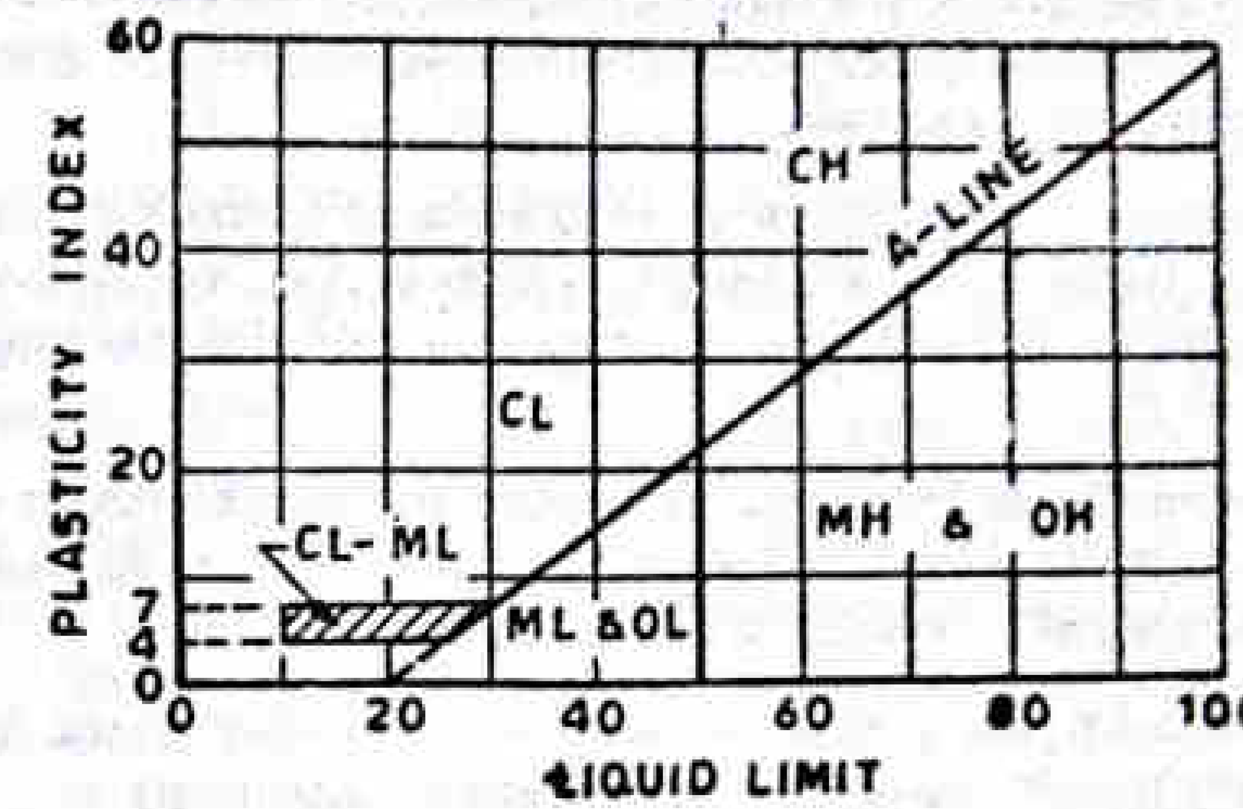



**Fig. 6.2 Plasticity Chart**

**Table 6.2 I. S. Soil Classification**

| Division and | Sub-Division | Soil Group Description | Symbol |
|---|---|---|---|
| Coarse - grained soil (more than half of the total material larger than IS sieve 8) | Gravelly soils more than half grains larger than IS sieve 480 | Well-graded gravel or gravel sand mixtures, with clay binder | GB |
| | | Well-graded gravels or gravel sand mixtures with little or no fines. | GW |
| | | Clayey gravels poorly graded gravel-sand-clay mixtures | GC |
| | | Silty-gravel or poorly graded gravel-sand-silt mixtures | GM |
| | | Poorly graded gravels or gravel sand mixtures with little or no fines | GP |
| | Sandy soils, more than half of the coarse grains smaller than IS sieve 480 | Well-graded sand or gravelly sands, with clay binder | SB |
| | | Well-graded sand or gravelly sands with little or no fines | SW |
| | | Clayey sand poorly graded sand-clay mixtures | SC |
| | | Silty sands or poorly graded sand-silt mixtures | SM |
| | | Poorly graded sands or gravelly sands with little or no fines | SP |
| Fine grained inorganic soils (more than half of the total material smaller than IS sieve 8) | Inorganic silts and clays with low or Medium compressibility | Silt and very fine sand; rock flour; silty to clayey fine sands with low plasticity | ML |
| | | Gravelly clays, sandy clays, silty clays, lean clays of low plasticity | CL |
| | | Clay of medium plasticity | CI |
| | Inorganic silts and clays with high compressibility | Very compressible micaceous or diatomaceous fine silty soils, silts | MH |
| | | Clays of high plasticity | CH |
| Silt and clay with high organic content | Silts, organic | Silty and Silt-clays of low plasticity | OL |
| | Clays, organic | Clays of medium to high plasticity very compressible | OH |
| Peat | Peat | Peat and other highly organic swamp soils | Pt |

## Highway Research Board (HRB) classification of soils

This is also called American Association of State Highway Officials (AASHO) classification of Revised Public Roads Administration (PRA) soil classification system. The original soil classification was developed by Bureau of Public Roads in 1928. After about 15 years of experience, certain revisions and modifications were made and the HRB classification system was developed. In fact by these modifications it is now possible to have subgroups for properly classifying different soil types and the number of classification tests were decreased from six to three, thus enabling classification of soils by three simple laboratory tests namely, sieve analysis liquid limit and plastic limit.

Soils are divided into seven groups A-1 to A-7. A-1, A-2 and A-3 soils are granular soils, percentage fines passing 0.074 mm sieve being less than 35. A-4, A-5, A-6 and A-7, soils are fine grained or silt-clay soils, passing 0.074 mm sieve being greater than 35 percent.

A-1 soils are well graded mixture of stone fragments, gravel coarse sand, fine sand and non-plastic or slightly plastic soil binder. The soils of this group are subdivided into two subgroups, A-1-a, consisting predominantly of stone fragments or gravel and A-1-b consisting predominantly of coarse sand.

A-2 group of soils include a wide range of granular soils ranging from A-1 to A-3 groups, consisting of granular soils and upto 35% fines of A-4, A-5, A-6 or A-7 groups. Based on the fines content, the soils of A-2 groups are subdivided into subgroups A-2-4, A-2-5, A-2-6 and A-2-7.

A-3 soils consist mainly, uniformly graded medium or fine sand similar to beach sand or desert blown sand. Stream-deposited mixtures of poorly graded fine sand with some coarse sand and gravel are also included in this group.

A-4 soils are generally silty soils, non-plastic or moderately plastic in nature with liquid limit and plasticity index values less than 40 and 10 respectively.

A-5 soils are also silty soils with plasticity index less than 10%, but with liquid limit values exceeding 40%. These include highly elastic or compressible, soils, usually of diatomaceous of micaceous character.

A-6 group of soils are plastic clays, having high values of plasticity index exceeding 10% and low values of liquid limit below 40%; they have high volume change properties with variation in moisture content.

A-7 soils are also clayey soils as A-6 soils, but with high values of both liquid limit and plasticity index, (LL greater than 40% and PI greater than 10%). These soils have low permeability and high volume change properties with changes in moisture content.

*Groups Index of Soil*

Fine grained soils of each classification group exhibit a wide range of properties as subgrade material.

In order to classify the fine grained soils within one group and for judging their suitability as subgrade material, an indexing system has been introduced in HRB classification which is termed as *Group Index.* Soils are thus assigned arbitrary numerical numbers known as group index (GI). Group index is function of percentage material passing 200 mesh sieve (0.074 mm), liquid limit and plasticity index of soil and is given by the equation :

$$\text{G I} = 0.2\,a + 0.005\,a\,c + 0.01\,bd \qquad (6.1)$$

Here, a = that portion of material passing 0.074 mm sieve, greater than 35 and not exceeding 75 percent (expressed as a whole number from 0 to 40)

b = that portion of material passing 0.074 mm sieve greater than 15 and not exceeding 35 percent (expressed as a whole number from 0 to 40)

c = that value of liquid limit in excess of 40 and less than 60 (expressed as a whole number from 0 to 20)

d = that value of plasticity index exceeding 10 and not more than 30 (expressed as a whole number from 0 to 20)

According to this formula, the minimum possible value of group index is zero and the maximum possible value is 20, when the values of soil fraction passing 0.074 mm sieve, liquid limit and plasticity index are respectively higher than 75, 60 and 30 percent. Higher the value of group index, poorer is the soil as subgrade material.



The sub-groups A-2-6 and A-2-7 soils of A-2 group have GI values 0 to 4, A-4 group of soil have GI values up to 8; A-5 soil up to 12, A-6 soil up to 16 and A-7 soil up to 20. The group index value is indicated as suffix to the soil group within brackets, such as A-6 (4) or A-6 (16). In this example an A-6 soil with group index value 4 is considered as superior subgrade material than the A-6 soil with group index value 16.

The soil groups showing the classification limits of various properties is given in Table 6.3. In order to classify a soil, the values of test results are attempted to be fitted in from left column towards right side of Table 6.3 and the correct group is found by the process of elimination. The first group from the left to which the test data fits in gives the classification group. Figure 6.3 gives the chart for finding the group index value form the values of percent passing 0.074 mm sieve, liquid limit and plasticity index, instead of using the Eq. 6.1 for group index. Here the group index value is the sum of the values obtained on vertical axes from both the charts based on the values of percent passing 0.074 mm sieve, LL and PL Fig. 6.4 gives the chart for classifying fine grained soil from the liquid limit and plasticity index values. GI values have been made use of in the design of flexible pavement thickness in one of the empirical design methods as given in Art. 7.3.2.

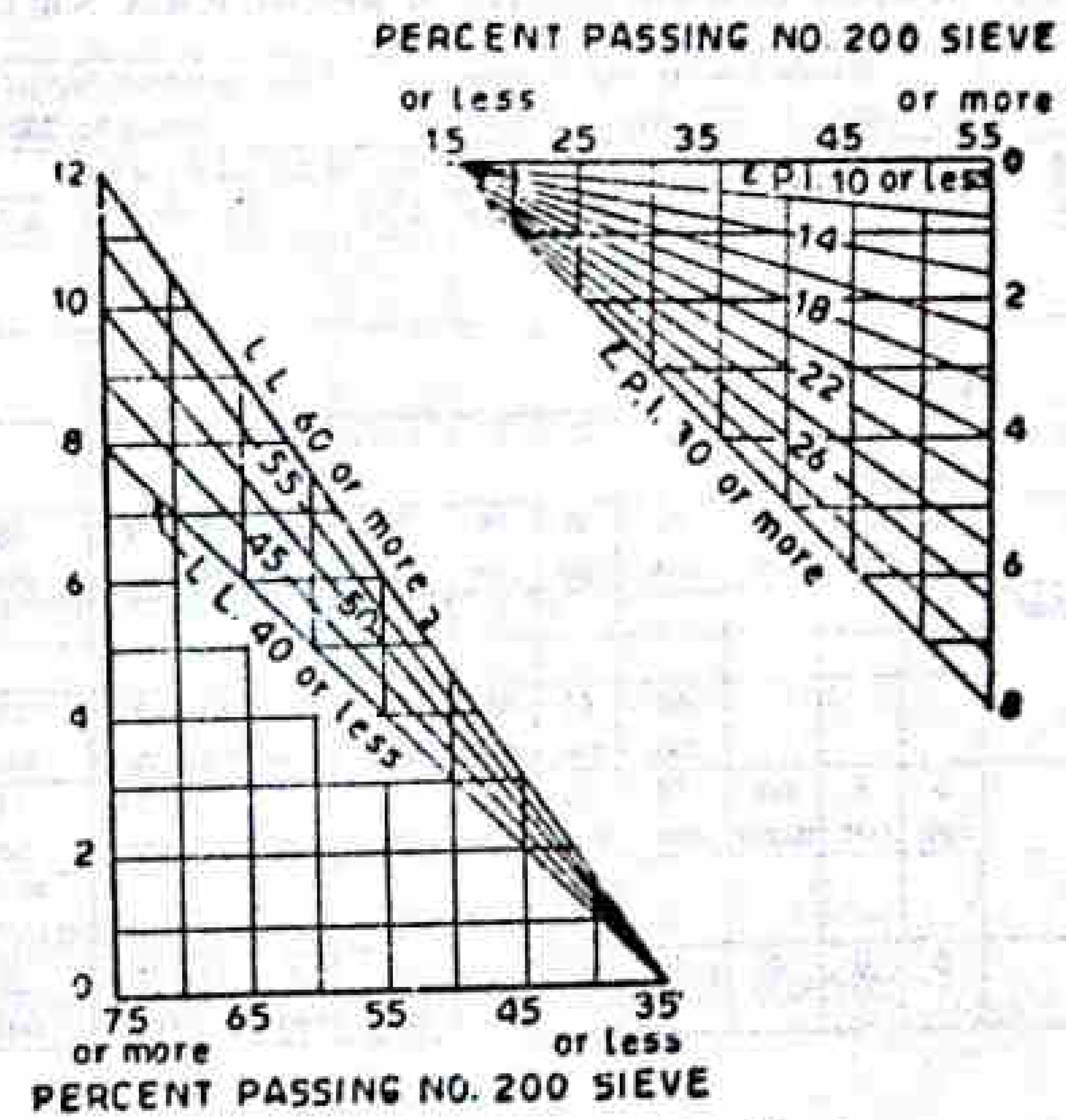



**Fig. 6.3 Group Index Chart**

*Characteristics of Soil Classification Groups*

The soil classification systems have gained importance in subsequent years, as the soils belonging to the different classification groups are qualified based on their stability or bearing capacity, drainage characteristics, potential frost action and volume change properties. The suitability of soils as subgrade material could also be judged by knowing the soil classification group. In Table 6.4, some of these properties of the unified soil classification groups are given. Table 6.5 gives some of the properties of the HRB soil classification groups. Thus soil classification is useful to predict some of the soil characteristics from simple physical tests.

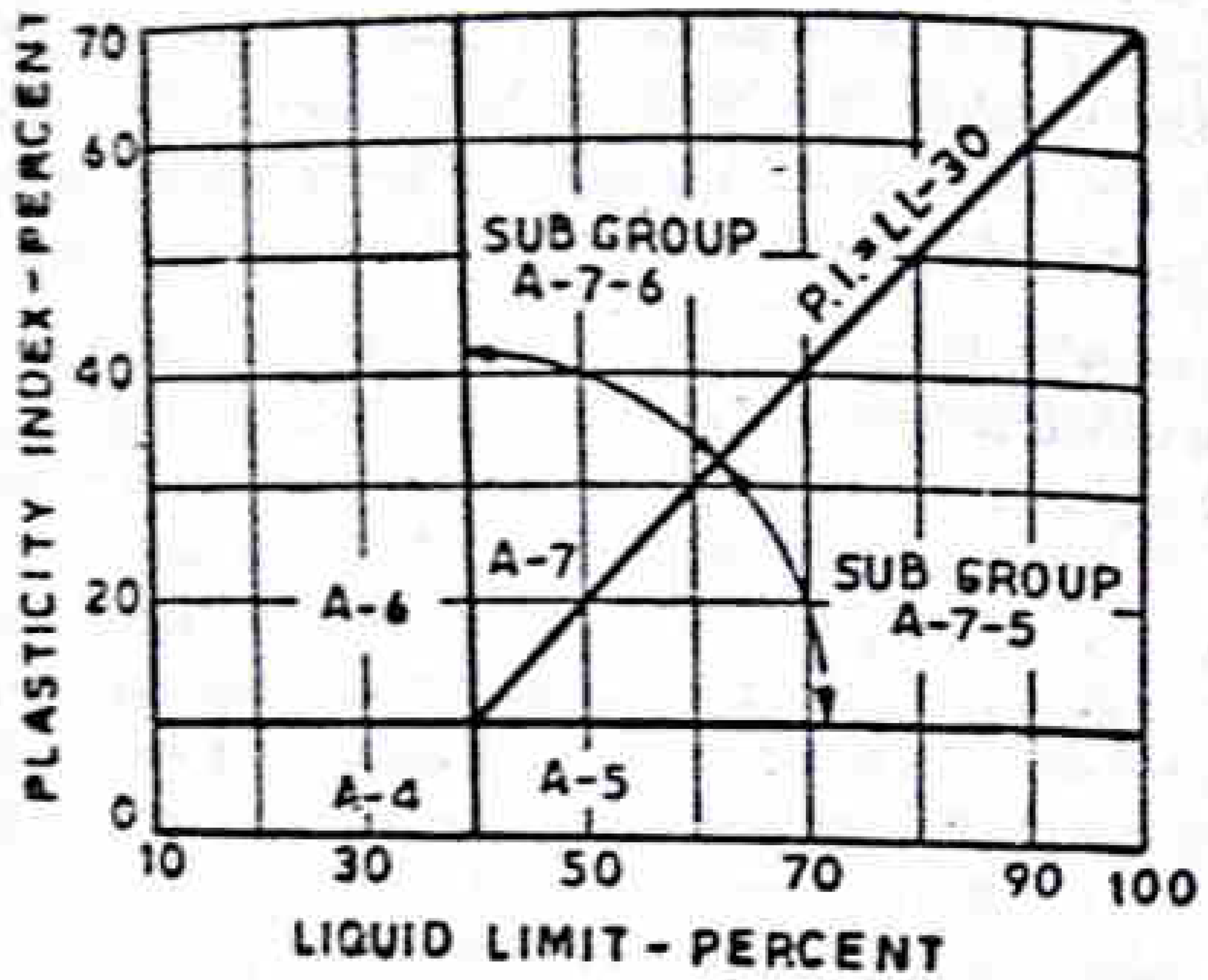



Fig. 6.4 Chart for Classifying Fine Grained Soil (H.R.B. system)

Table 6.3 Highway Research Board or AASHO or Revised P.R.A. Soil Classification

| General Classification | Granular soils, less than 35 percent passing No. 200 sieve size (0.074 mm) | | | | | | | Fine grained (silt-clay) soils, more than 35% passing No. 200 sieve size | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Classification Group | A-1 | | A-3 | A-2 | | | | A-4 | A-5 | A-6 | A-7 | |
| Sub classification | A-1-a | A-1-b | | A-2-4 | A-2-5 | A-2-6 | A-2-7 | | | | A-7-5 | A-7-6 |
| Sieve analysis, % passing | | | | | | | | | | | | |
| No. 10 sieve (2 mm size) | 50 max. | | | | | | | | | | | |
| No. 40 sieve (0.42 mm size) | 30 max. | 50 max. | 51 min. | | | | | | | | | |
| No. 200 sieve (0.074 mm size) | 15 max. | 25 max. | 10 max. | 35 max. | 35 max. | 35 max. | 35 max. | 36 min. | 36 min. | 36 min. | 36 min. | 36 min. |
| Characteristics of fraction Passing no. 40 sieve | | | | | | | | | | | | |
| Liquid limit | - | - | - | 40 max. | 41 min. | 40 max. | 41 min. | 40 max. | 41 min. | 40 max. | 41 min. | 41 min. |
| Plasticity index | 6 max. | 6 max. | non plastic | 10 max. | 10 max. | 11 min. | 11 min. | 10 max. | 10 max. | 11 min. | 11 min. | 11 min. |
| | | | | | | | | | | | PI < (LL − 30) | PI > (LL − 30) |
| Group Index | 0 | 0 | 0 | 0 | 0 | 4 max. | 4 max. | 8 max. | 12 max. | 16 max. | 20 max. | 20 max. |

**Example 6.1**

The results of sieve analysis of a soil are given below :

| Sieve size, mm | Percent passing |
|---|---|
| 4.76 | 60 |
| 2.00 | 30 |
| 0.60 | 10 |
| 0.40 | 5 |
| 0.20 | 0 |

(a) Classify the soil by (i) Unified and (ii) HRB soil classification systems.

(b) Discuss the suitability of the soil as a subgrade material.



**Solution**

(a) *Soil classification*

(i) By Unified System

Refer Table 6.1. As more than half (60%) is finer than 4.76 mm sieve and there are no fines the soil is sand or SW or SP groups.

$$\text{Uniformity coefficient } C_u = \frac{D_{60}}{D_{10}} = \frac{4.76}{0.6} = 7.9$$

$$\text{Gradation coefficient } C_g = \frac{(D_{30})^2}{D_{60}\,D_{10}} = \frac{2.0^2}{4.76 \times 0.6} = 1.7$$

As $C_u$ is greater than 4 and $C_g$ is between 1 and 3, the classification group of the sand is SW.

(ii) By HRB System

From Table 6.3, it is found that the classification group is A-1-a.

*(b) Suitability as Subgrade Material*

Refer Tables 6.4 and 6.5

| | |
|---|---|
| Drainage | excellent |
| Volume change | almost none |
| Potential frost action | none to very slight |
| Stability | high |
| Values as subgrade material | very good |

**Example 6.2**

The properties of subgrade soil are given below :

Passing 0.074 mm sieve = 55%

Liquid limit = 50%

Plastic limit = 41%

(a) Classify the soil by revised PRA/HRB system

(b) Discuss the suitability of the soil as a subgrade material.

**Solution**

*(a) Soil Classification*

From Table 6.3, as percentage passing 0.074 mm sieve is more than 35, the soil is fine grained.

$$LL = 50\%$$

$$PI = LL - PL = 50 - 41 = 9$$

Using the chart (Fig. 6.4) the classification group is A-5.

$$\text{Group Index, GI} = 0.2\,a + 0.005\,ac + 0.01\,bd$$

**Table 6.4 Some Characteristics of unified Soil Classification Groups**

| Soil Type | Soil Group | Value as foundation or subgrade material | Unit dry weight (I.S. light compaction) g/m³ | CBR % | Subgrade modulus kg/cm³ | Drainage characteristics | Volume change characteristics | Potential frost action |
|---|---|---|---|---|---|---|---|---|
| Coarse grained soils Gravelly soils (G) | GW | Excellent | 2.00 – 2.24 | 60 – 90 | > 8.33 | Excellent | Almost none | None to very slight |
| | GP | Excellent to good | 1.76 – 2.08 | 25 – 60 | > 8.33 | Excellent | None | None |
| | GM | Excellent to good | 2.08 – 2.22 | 20 – 80 | > 8.33 | Fair to poor | Very slight | Slight to medium |
| | GC | Good | 1.92 – 2.24 | 20 – 80 | 5.55 – 8.33 | Poor | Slight | Slight to medium |
| Coarse grained soils-Sand soils (S) | SW | Good | 1.76 – 2.08 | 20 – 60 | 5.55 – 8.33 | Excellent | Almost none | None to very slight |
| | SP | Good to fair | 1.59 – 1.92 | 10 – 30 | 5.55 – 8.33 | Excellent | Almost none | None to very slight |
| | SM | Good | 1.92 – 2.16 | 10 – 40 | 5.55 – 8.33 | Fair to poor | Very slight | Slight to high |
| | SC | Fair to Good | 1.68 – 2.08 | 15 – 50 | 5.55 – 8.33 | Poor | Slight to medium | Slight to high |
| Fine grained soils with low Comp - ressibility | ML | Fair to Poor | 1.60 – 2.00 | 5 – 20 | 2.78 – 5.55 | Fair to poor | Slight to medium | High to very high |
| | CL | Fair to Poor | 1.60 – 2.00 | 5 – 15 | 2.78 – 5.55 | Very poor | Medium | Medium to high |
| | OL | Poor | 1.44 – 1.60 | 3 – 8 | 2.78 – 5.55 | Poor | Medium to high | Medium to high |
| Fine grained soils with high comp - ressibility | MH | Poor | 1.28 – 1.60 | 3 – 8 | 2.78 – 5.55 | Poor | High | Medium to high |
| | CH | Poor to very poor | 1.44 – 1.76 | 3 – 5 | 1.39 – 2.78 | Very poor | Very High | Medium |
| | OH | Poor to very poor | 1.28 – 1.68 | 2 – 4 | 1.39 – 2.78 | Very poor | Very High | Medium |

$$a = 55 - 35 = 20$$

$$b = 55 - 15 = 40$$

$$c = 50 - 40 = 10$$

$$d = 9 - 10 = 0 \text{ (min)}$$

$$\therefore GI = 0.2 \times 20 + 0.005 \times 20 \times 10 + 0.01 \times 40 \times 0$$

$$= 4.0 + 1.0 + 0 = 5.0$$

Alternatively GI may also be found using charts (Figure 6.3) and is found to be GI = 5.0 + 0 = 5.0.

Soil classification is A – 5 (5).

*(b) Suitability as subgrade material*

From Table 6.5,

| | |
|---|---|
| Drainage | poor |
| Volume change | medium to high |
| Potential frost action | high to very high |
| Stability | poor |
| Values as subgrade material | poor |



**Fig. 6.5 Some Characteristics of H.R.B. Soil Classification Groups and Approximate Equivalent Group of Soil Classification System**

| Soil Group | Sub Group | General Stability Property and rating as subgrade material | Max. dry density (I. S. Light comp c/um³) | CBR % | Subgrade modulus kg/cm³ | Drainage characteristics | Volume change characteristics | Potential frost action | Approximate equivalent unified classification |
|---|---|---|---|---|---|---|---|---|---|
| A-1 | A-1-a | High stability very good to excellent subgrade | 2.03 (min) | 60 – 90 | > 8.33 | Excellent | Almost none | None to slight | GW, GP, GM |
| | A-1-b | | | 20 – 70 | | Good | | | SW, SM |
| A-3 | | Stable when confined very good to fair subgrade material | 1.29 – 2.03 | 10 – 80 | > 5.55 | Excellent | None | None | SP |
| A-2 | A-2-4 | Stable when dry; may ravel. Very good to good subgrade material | 1.92 – 2.08 | 8 – 70 | > 5.0 | Good to Fair | Very slight | Slight to high | GM, SM |
| | A-2-5 | | | | | | | | |
| | A-2-6 | Good stability. Very good to fair subgrade material | | | | Fair to poor | Slight to medium | Very slight to medium | GC, SC |
| | A-2-7 | | | | | | | | |
| A-4 | | Satisfactory stability when dry. Loss of stability when wet or by frost action. Good to poor subgrade material | 1.76 – 1.29 | 4 – 20 | 2.78 – 5.0 | Fair to poor | Slight to medium | Very high | ML, OL |
| A-5 | | Difficult to compact doubtful stability, poor to very poor subgrade material | 1.28 – 1.60 | 2 – 7 | 1.39 – 3.48 | Poor | Medium to high | High to very high | MH |
| A-6 | | Good stability when compacted in unsoaked condition. Fair to poor subgrade material | 1.28 – 1.76 | 2 – 15 | 1.39 – 5.55 | Very poor | High | Medium to high | CL |
| A-7 | A-7-5 | Good stability when properly compact & in unsoaked conditions; poor subgrade material when wet. | 1.28 – 1.76 | 2 – 15 | 1.39 – 5.55 | Very poor | Very high | Medium | CL, OL, CH, OH |
| | A-7-6 | | | | | | | | |

### 6.1.7 Subgrade Soil Strength

The factor on which the strength characteristics of soil depend are :

(i) soil type

(ii) moisture content

(iii) dry density

(iv) internal structural of the soil, and

(v) the type and mode of stress application

The problem of predicting the stress-strain relationship of soil is difficult, because of the diversity in the soil types and the non-homogeneous nature of the soil under the foundations. Generally the highway engineer is interested in the stability or the resistance to deformation of the soil under the stress applications.

In a soil mass, the deformation is largely due to slippage between soil particles. Hence the *shearing resistance* in soil represents the strength. The sliding mechanism in soils is complicated as the shear deformation cause reorientation of particles resulting in changes in volume, valence bond between particles, thickness and other properties of adsorbed layer of water.

Though many theories of failures of materials are known, *Mohr's theory* is the most useful one for soils. The basis of his theory is that a material fails when the shearing stress on the failure plane is definite function of the normal stress acting on that plane and that failure occurs by slippage only. Shearing resistance in a soil mass is commonly attributed to internal *friction* and *cohesion* parameters of the soil. For majority of soils shearing resistance is made up of both friction and cohesion. For these soils the shearing resistance on any plane is given by *Coulomb's* empirical law :

$$S_r = C + \sigma \tan \phi \qquad (6.2)$$

Here C is cohesion per unit area, $\phi$ is the angle of internal friction and $\sigma$ is the normal stress. The value of $\phi$ depends on the dry density of the soil, grain size distribution, shape and texture of soil strength hence depends on the value of $\phi$ and the normal pressure on sliding plane, $\sigma$. Cohesion C is the resistance of soil grains to displacement by bond developed at the surface of contact by very fine grained soils as result of intermolecular and electrochemical forces of attraction. Cohesion may be said to include, in simple terms, both the *true cohesion* which is due to intermolecular attraction and the *apparent cohesion* which is due to surface tension effects of the held water. The value of C depends on the type of clay mineral, its size, the surface charges, the proportion of the clay and the water content.

The stability of a soil or its strength property is often determined from its stress-deformation characteristics. This depends very much on the type of stress application, its intensity and rate of application. In case of static stresses the period of stress application and the intensity of stress application have significant effect on soils which show *viscoelastic behaviour*. In case of repeated application of stresses, the frequency of loading cycle, the magnitude of stress and the number of repetitions have influence.

### 6.1.8 Evaluation of Soil Strength

The tests used to evaluate the strength properties of soils may be broadly divided into three groups :



(i) Shear tests,

(ii) Bearing tests, and

(iii) Penetration tests

There are number of test methods in each group.

*Shear tests* are usually carried out on relatively small soil samples in the laboratory. In order to find the strength properties of a soil, a number of representative samples from different locations are tested. Some of the commonly known shear tests are direct shear test, triaxial compression test and unconfined compression test. Vane shear tests may be carried out either on a soil sample or in-situ soil in the field.

*Bearing tests* are loading tests carried out on subgrade soils in situ with a load bearing area. The results of the bearing tests are influenced by the variations in the soil properties within the stressed soil mass underneath and hence the overall stability of the part of the soil mass stressed could be studied.

*Penetration tests* may be considered as small scale bearing tests in which the size of the loaded area is relatively much smaller and ratio of the penetration to size of loaded area is much greater than the ratios in bearing tests. The penetration tests are carried out in the field or in the laboratory. The *California Bearing Ratio* test and cone penetration tests are commonly known penetration tests.

There are number of factors which affect the results of the strength tests as mentioned below :

(i) Factors which are primarily associated with the actual tests such as size and shape of the specimen, method of loading, rate of loading and drainage conditions.

(ii) Factors which are associated with the soil such as soil type dry density, moisture content, permeability structure and other properties of the soil

Some of the commonly adopted tests for evaluating soil strength characteristics are briefly discussed here.

**Direct shear test**

This is one of the oldest of shear tests. The direct shear apparatus consists essentially of a box divided horizontally into two halves. One half is kept fixed and the other half is free to move horizontally. A vertical load is applied and the horizontal pull is caused to produce a certain rate of horizontal displacement. The vertical and horizontal movements are measured by dial gauges and the horizontal force is noted from the proving ring dial. The maximum horizontal force is measured for different values of normal load. The values of maximum shear stress applied for the different vertical stresses are computed and plotted as shown in Figure 6.5. The values of cohesion (C) and angle of internal friction ($\phi$) are found using Coulomb's equation for shear stress (Equation 6.2).

There are a number of limitations of this test. The failure plane being predetermined horizontal plane, need not necessarily represent the *imminent plane* of failure. The shearing stress and strain along this horizontal failure plane is seldom uniform. The area of cross section of specimen decreases with displacement. The flow of water to or from the soil specimen can not be easily controlled or measured. It is also not practicable to measure pore water pressure during the test. Carrying out an undrained (quick) test on a sandy soil is rather impracticable in the shear box.

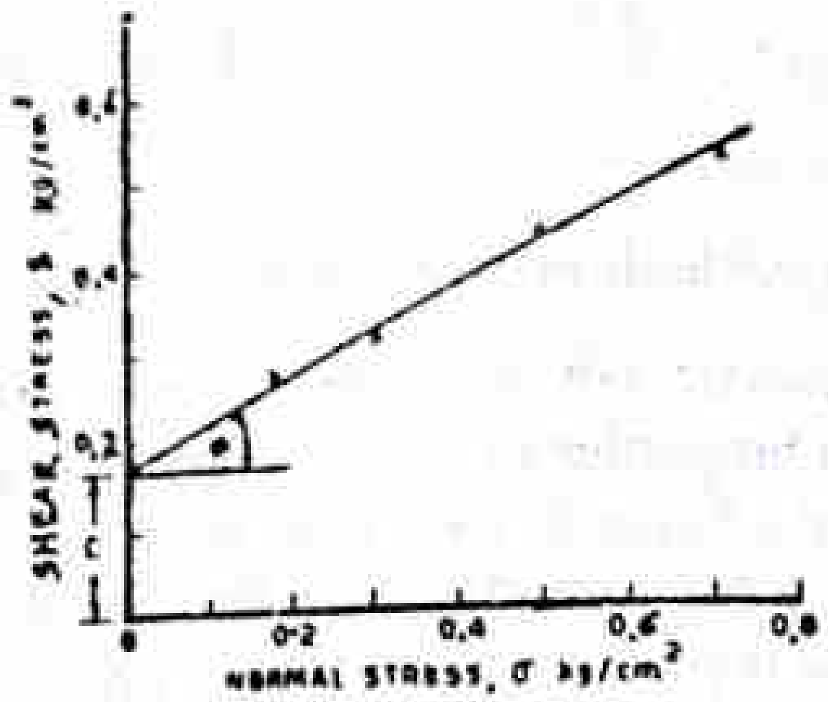



**Fig. 6.5 Direct Shear Test Results**

**Triaxial compression test**

The object of triaxial compression test is to determine the shear strength of soil under *lateral confinement*. An attempt is made to simulate the confining pressure observed in a loaded soil mass. In the test set up, it is possible to provide uniform fluid-confining pressure only. Cylindrical specimen of height to diameter ratio 2 is inserted in a thin membrane, placed in a triaxial cell and the required lateral pressure is applied. The normal load is applied through vertical piston by means of a strain controlled machine and the maximum vertical load at failure is recorded. The specimens are usually subjected to a constant strain rate of 1.25 mm per minute. Usually the lateral pressure, $\sigma_3$ is maintained constant and the vertical pressure, $\sigma_1$ is increased until the specimen fails. In some studies the volume of the specimen is maintained constant by adjusting the lateral pressure, $\sigma_3$ during application of vertical stress, $\sigma_1$. The specimen may either fail by shearing or in the case of saturated clayey soils by *bulging*. The deviator stress, $\sigma_d$ under which the specimen fails is $(\sigma_1 - \sigma_3)$. (See Fig. 6.6). The various values of normal stress, $\sigma_1$ and hence deviator stress, $\sigma_d$ corresponding to the different values of lateral pressure $\sigma_3$ are obtained from the triaxial tests.

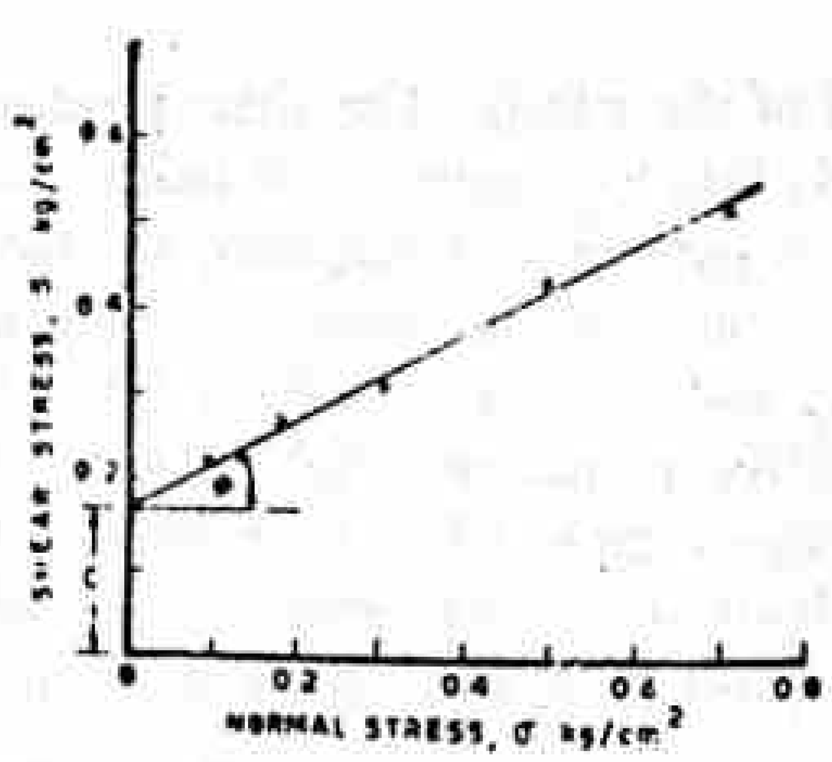



**Fig. 6.6 Stresses in Triaxial Test**

The tests are carried out at different lateral pressures preferably with atleast three lateral pressures. For the pavement design, lateral pressure of 0, 0.75 and 1.5 kg/cm$^2$ are considered desirable. The values of confining pressure and total vertical pressure at failure are plotted and semicircles passing through these points are drawn as shown in



Fig. 6.7. A common line tangential to the circles is drawn, representing the *Mohr rupture envelope*. The intercept of this line with Y-axis represents the cohesion C and the inclination with X-axis represents the angle of internal friction $\phi$ of the soil.

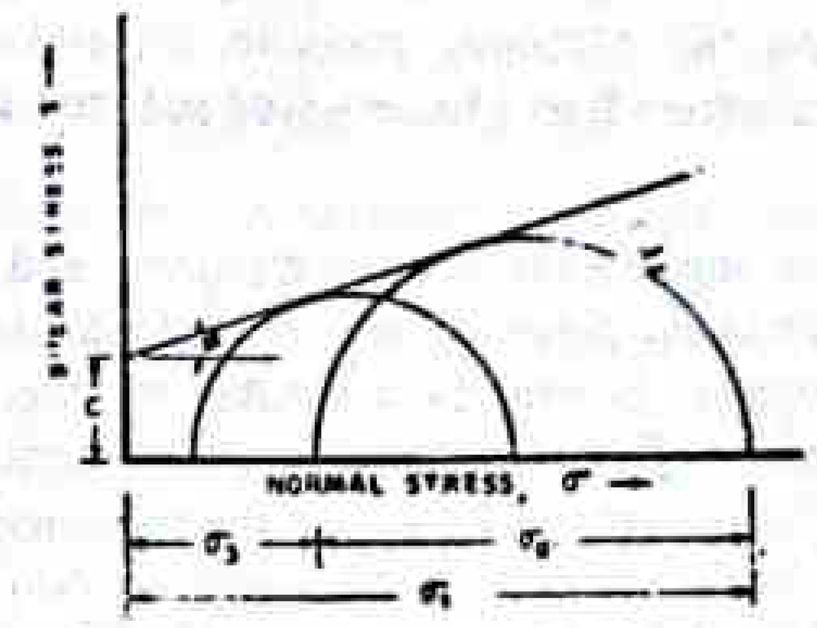



**Fig. 6.7 Triaxial Test Results**

*Correction for Area of Cross Section*

In is necessary to correct the stress value for the increased area of cross section due to loading. The corrected value of deviator stress is found with the assumption that the volume of the specimen remains constant and the area of cross section of the specimen is uniformly increased from the original value of $A_o$ to $A_1$ due to load $P_1$. Volume $A_o l_o = A_1 l_1 = A_1 (l_o - \Delta)$. In that case the deviator stress $\sigma_d$ is given by :

$$\sigma_d = \frac{P_1}{A_1} = \frac{P_1 . l_1}{A_o l_o} = \frac{P_1}{A_o} = \left(\frac{l_1 - \Delta}{l_o}\right)$$

i.e.

$$\sigma_d = \frac{P_1}{A_o}(1-\delta) \qquad (6.3)$$

Here $\Delta$ = total deformation of the specimen = $(l_o - l_1)$

$\delta$ = unit strain = $\Delta / l_o$

$A_o$ = original area of cross section

$P_1$ = applied load

$\sigma_d$ = deviator stress.

After calculating the corrected value of $\sigma_d$, $\sigma_1$ is calculated from the relation, $\sigma_1 = \sigma_d + \sigma_3$.

*Basic Types of Triaxial Tests*

The behaviour of the soil specimen during testing is influenced by the condition of drainage allowed during the test.

Drained tests or slow tests are those in which the drainage of specimen is permitted during the application of both stresses, $\sigma_3$ and $\sigma_d$. The specimen is allowed to fully consolidate under the confining pressure, $\sigma_3$ and then the deviator stress, $\sigma_d$ is applied and increased so slowly that no significant pore pressure is built up while the specimen is under the test.

In consolidated-undrained test or consolidated-quick test, complete consolidation is allowed under the confining pressure $\sigma_3$ before applying deviator stress. No drainage is permitted when the deviator stress is applied.

In undrained or quick test, no drainage is allowed at any stage. The drainage is prevented before applying the confining pressure, $\sigma_3$ and during the application of deviator stress until the specimen fails. Excess pore pressure commonly exist throughout the test.

In pavements the load applied are mostly *transient* and during the loading cycle drainage can not take place in the subgrade soil. In order to simulate consolidation of the subgrade under the pavement, it may be desirable to consolidate the sample to an equivalent confining pressure before the application of the load and thus the consolidated quick test is justified for the pavement design. If the pavement is designed for sustained loading conditions as for the parked or stationary vehicle, then the possibility of drainage in the subgrade during the loaded period may be considered.

*Interpretation of Results of Triaxial Test*

Besides values of C and $\phi$ the behaviour of the soil under the loading conditions is judged from the stress-strain relationship of the soil as shown in Figure 6.8. These plots seldom indicate a straight portion even at the initial stages of stressing. Hence it is not possible to interpret the strength of soil in terms of modulus of elasticity as done in the case of various structural materials. In flexible pavement design the *modulus of deformation* of the soil is used. The modulus of deformation is the ratio of stress to strain at an arbitrary point on the stress-strain curve. This point is decided based on the anticipated stress in the subgrade under the pavement or an allowable value of deformation. As an example, if the anticipated stress under a flexible pavement is p kg/cm$^2$, then the modulus of deformation is equal to p/$\delta$ where $\delta$ is deformation corresponding to a stress of p kg/cm$^2$, in the triaxial test. Now the value of the modulus of deformation at any stress level will depend on the confining pressure $\sigma_3$ applied. Hence it becomes necessary to decide the value of confining pressure for the triaxial test. However the term modulus of elasticity is also used instead of modulus of deformation for usual computations.

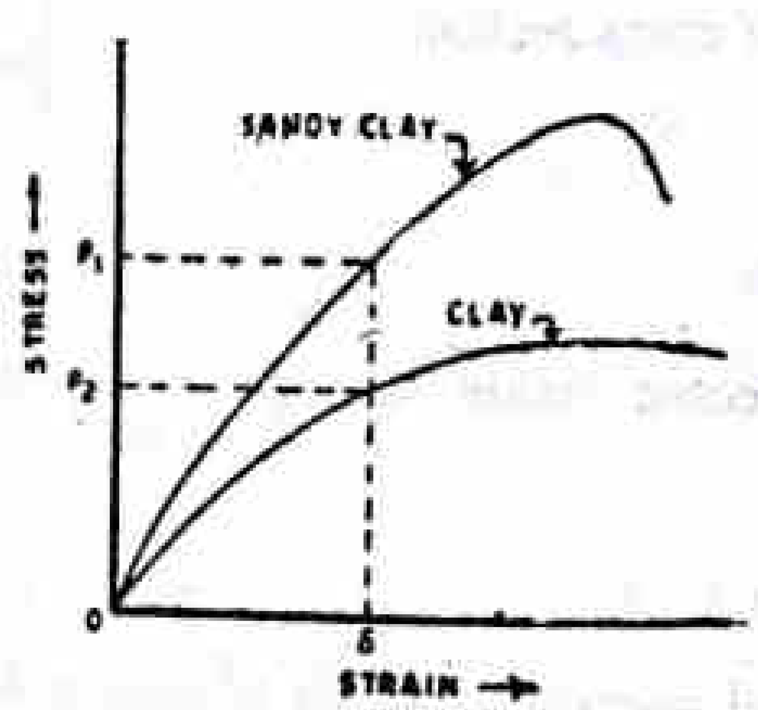



**Fig. 6.8 Stress-Strain Relationship of soils**

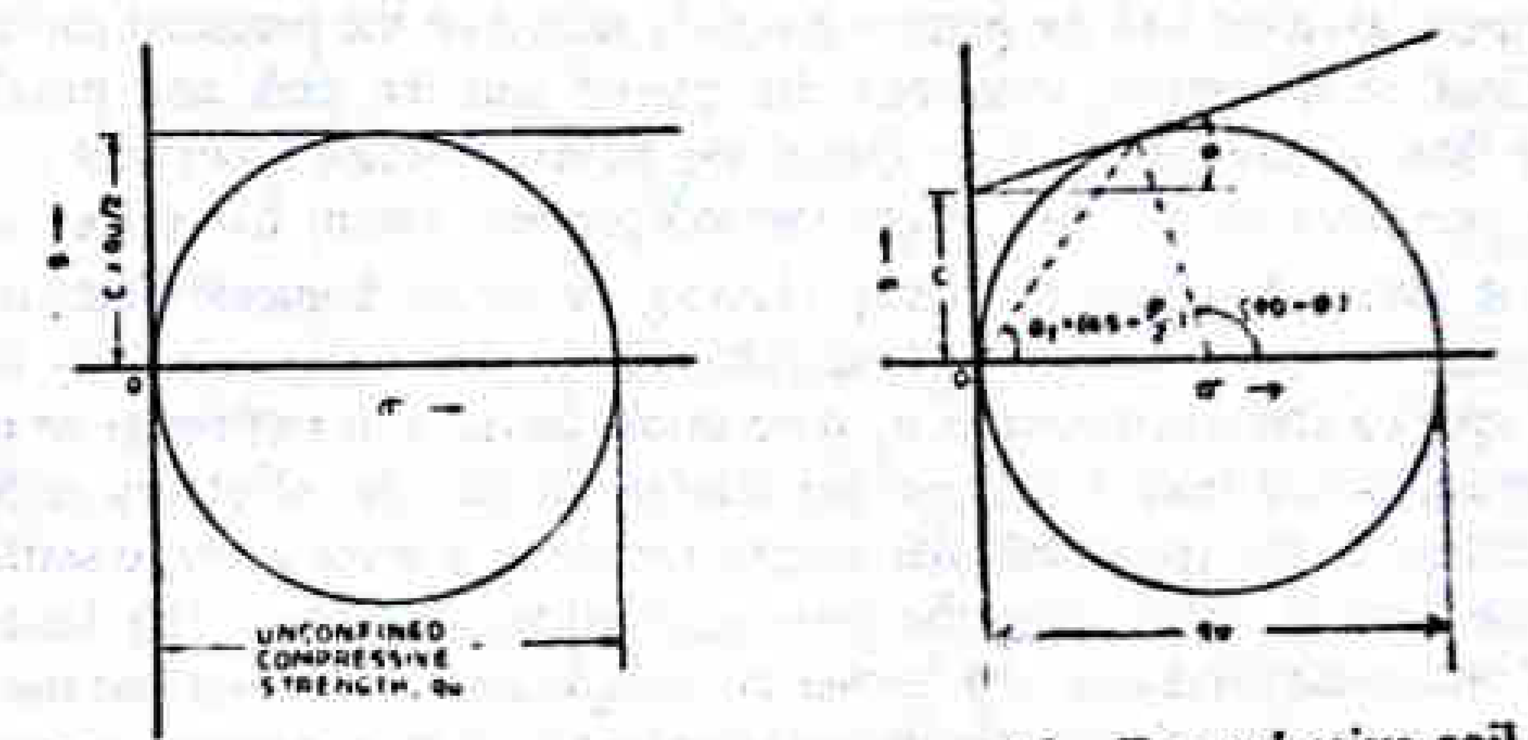
**Unconfined compression test**

The unconfined compression test may be considered as a special case of triaxial compression test when the confining pressure is zero and the axial compressive stress only is applied to the cylindrical specimen. The stress may be applied and the



deformation and load readings are noted until the specimen fails. The area of cross section of specimen for various strains may be corrected assuming that the volume of the specimen remains constant and that the specimen retains cylindrical cross sectional areas as explained under triaxial test. The maximum axial strain is noted. In clays when plastic failure takes place, no definite value of maximum or peak stress could be noticed; in such cases the stress at 20 percent strain is some time adopted.

The Mohr circle of rupture for an unconfined compression test passes through the origin. It is not possible to draw the Mohr rupture envelope from a single circle, and so the values of C and $\phi$ cannot be determined as such. In case of quick or undrained test carried out on saturated clays, the value of $\phi$ may be assumed as zero and hence the Mohr envelope will be a horizontal line touching the circle with the unconfined compressive strength $q_u$ as diameter as shown in Fig. 6.9(a). Hence cohesion is half the unconfined compressive strength in this case i.e., $C = q_u/2$.




(a) For saturated clay where $\phi = 0$

(b) For cohesive soil

**Fig. 6.9 Unconfined Compression Test Results**

The failure plane of an unconfined compression or triaxial specimen (when a definite failure plane exists) makes an angle $\theta_f = (45 + \phi/2)$ with major principal plane. Hence if the angle made by the failure plane in the specimen with the horizontal, $\theta_f$, is noted, $\phi$ may be calculated. The value of C also can be calculated from the geometrical relationship of the Mohr diagram. The values of C and $\phi$ may be very easily found graphically as shown in Fig. 6.9 (b).

The unconfined compression or the simple compression is suitable to test cohesive soils and all materials having high values of cohesion or bond strength such as soil-cement etc.

**Plate bearing test**

The plate bearing tests is used to evaluate the supporting power of subgrade for use in pavement design by using relatively large diameter plates. The plate bearing test was originally devised to find the *modulus of subgrade reaction* in the Westergaard's analysis for wheel load stress in cement concrete pavements.

The test set up consists of a set of plates of diameter 75, 60, 45 and 30 cm, a loading device consisting of jack and proving ring arrangement and a reaction frame against which the jack can give a thrust to the plate. A datum frame resting far from the loaded area and dial gauges from this frame are used to measure the settlement of the loaded plate. The loading arrangement is shown in Fig. 6.10.

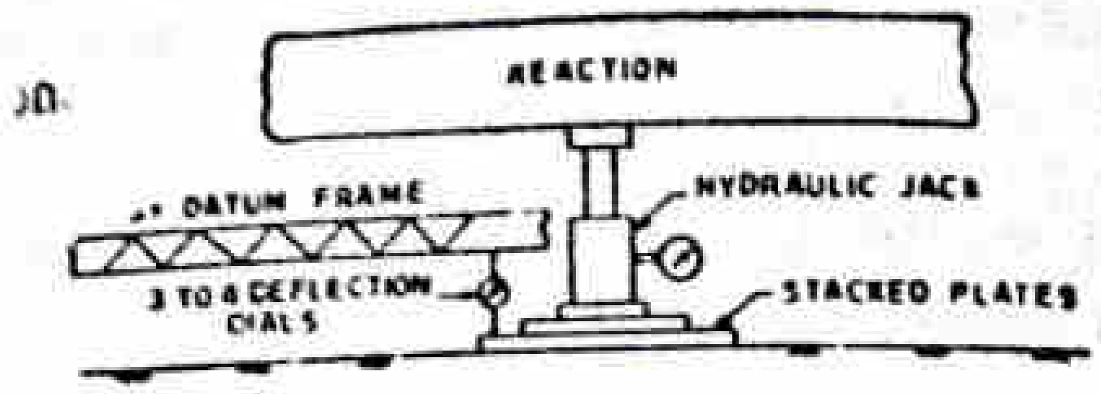



**Fig. 6.10 Plate Bearing Test Set Up**

*Modulus of Subgrade Reaction*

Modulus of subgrade reaction K may be defined as the pressure sustained per unit deformation of subgrade at specified deformation or pressure level, using specified plate size. The standard plate size for finding K-value is 75 cm diameter. But in some tests a smaller plate of 30 cm diameter is also used.

The test site is levelled and the plate is properly seated on the prepared surface. The stiffening plates of decreasing diameters are placed and the jack and proving ring assembly are fitted to provide reaction against the frame. Three or four dial gauges are fixed on the periphery of the plate, from the independent datum frame for measuring settlements. A seating load of 0.07 kg/cm$^2$ (320 kg for 75 cm diameter plate) is applied and released after a few seconds. A load sufficient to cause approximately 0.25 mm settlement is applied and 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 clayey soils or wet soils), the readings of the settlement dial gauges are noted and the average settlement is found, and the load is noted from the proving ring dial reading. The load is then increased till settlement increases to a further amount of about 0.25 mm and the average settlement and load are found. The procedure is repeated till the settlement reaches 0.175 cm. A graph is plotted with mean settlement versus mean bearing pressure as shown in Fig. 6.11. The pressure p corresponding to a settlement of 0.125 cm is read and the K-value is calculated by the reaction,

$$K = \frac{p}{0.125} \text{ kg/cm}^2\text{/cm (or kg/cm}^3\text{)} \qquad (6.4)$$

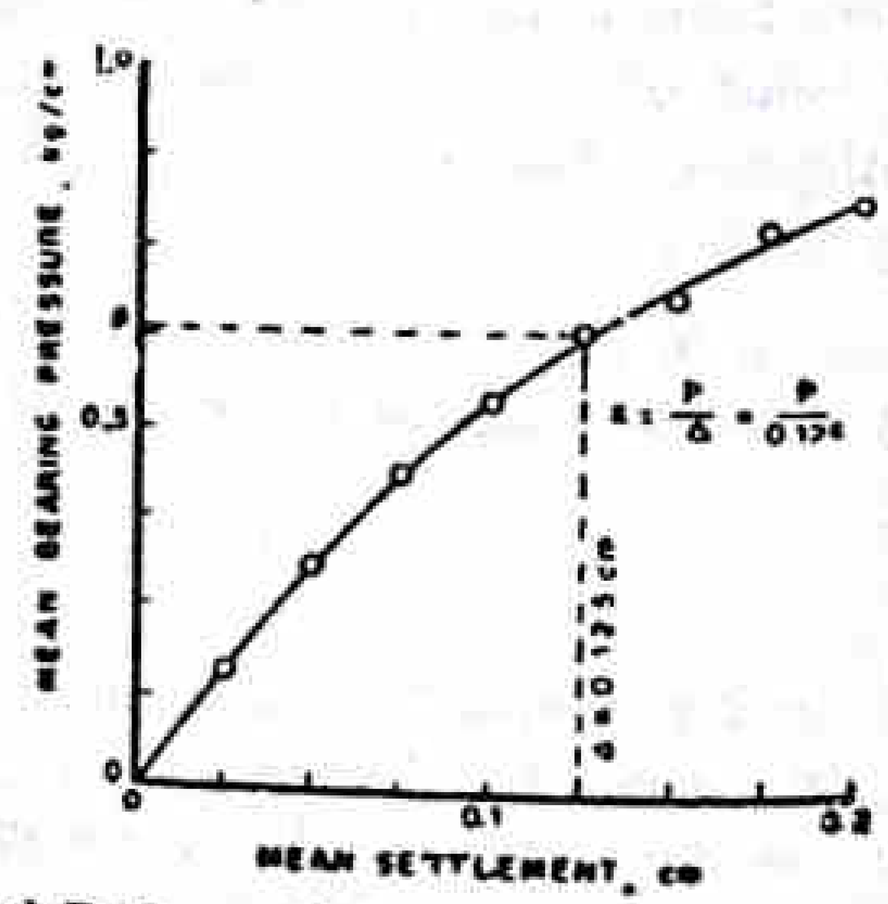



**Fig. 6.11 Load-Deformation Curve from Plate Bearing Test**

*Allowance for Worst Subgrade Moisture*

The moisture content at the time of carrying out plate bearing tests may seldom represent the worst moisture condition likely to occur at the test site. It may not be



practicable always to carry out the test at such a condition. In such cases the value of modulus of subgrade reaction K is found for the prevailing moisture content and the value so obtained may be modified for soaked condition.

After the plate bearing test, two consolidation test specimens are prepared. One specimen is tested as such without soaking by applying increments of pressure and the pressure deformation curve is plotted as shown in Fig. 6.12. The deformation $\delta$ of a sample corresponding to a pressure p kg/cm$^2$ required in the plate bearing test to cause a deformation of 0.125 cm is noted. Then the other specimen is soaked and consolidation test is carried out; the pressure $p_s$ required to produce the same deformation $\delta$ is noted, in the soaked test. (Refer Fig. 6.12). The modulus of subgrade reaction $K_s$ for the soaked condition is then calculated from the relation

$$K_s = K \frac{p_s}{p} \qquad (6.5)$$

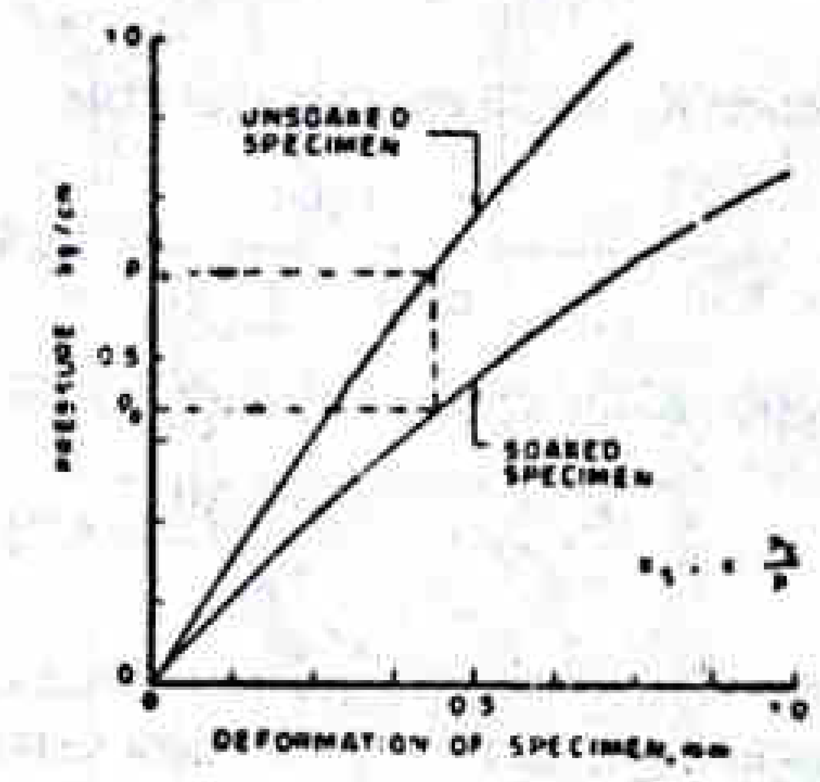



**Fig. 6.12 Correction for Soaking in Plate Bearing Test**

*Correction for Small Plate Size*

In some cases the load capacity may not be adequate to cause 75 cm diameter plate to settle 0.175 cm. In such case, a plate of smaller diameter (say 30 cm) may be used. Then K-value should be found by applying a suitable correction for plate size.

Assuming the subgrade to be an elastic medium with modulus of elasticity E (kg/cm$^2$) the theoretical relationship of deformation, $\Delta$ (cm) under a rigid plate of radius a (cm) is given by :

$$\Delta = 1.18 \frac{pa}{E} \qquad (6.6)$$

$$K = \frac{p}{\Delta} = \frac{p.E}{1.18\,p\,a} = \frac{E}{1.18\,a}$$

If the value of E is taken as constant for a soil then K is inversely proportional to *a* or Ka is constant i.e., Ka = $K_1 a_1$ or K = $K_1 a_1/a$. Hence if the test is carried out with a smaller plate of radius $a_1$ and the modulus of subgrade reaction $K_1$ is found, then the corrected value of modulus of subgrade reaction K for standard plate of radius a, is obtained from the relationship

$$K = K_1 \frac{a_1}{a} \qquad (6.7)$$

**Example 6.3**

A plate load test was conducted on a soaked subgrade during monsoon season using a plate diameter of 30 cm. The load values corresponding to the mean settlement dial readings are given below. Determine the modulus of subgrade reaction for the standard plate.

| Mean settlement values, mm | 0.0 | 0.24 | 0.52 | 0.76 | 1.02 | 1.23 | 1.53 | 1.76 |
|---|---|---|---|---|---|---|---|---|
| Load values kg | 0.0 | 460 | 900 | 1180 | 1360 | 1480 | 1590 | 1640 |

**Solution**

The load-settlement curve is plotted on a graph paper (similar to the one shown in Fig. 6.11) and the load value $p_1$ corresponding mean settlement value of $\Delta = 0.125$ cm is determined = 1490 kg.

$$\text{Unit load } p_1 = \frac{1490}{\pi 15^2} \text{ kg/cm}^2$$

Modulus of subgrade reaction $K_1$ for 30 cm diameter plate

$$= \frac{p_1}{\Delta} = \frac{1490}{\pi . 15^2 \times 0.125} = 16.86 \text{ kg/cm}^3$$

Modulus of subgrade reaction K for standard plate of dia. 70 cm.

$$= \frac{K_1 a_1}{a} = \frac{16.86 \times 30}{75} = 6.75 \text{ kg/cm}^3$$

(Note : As the plate load test was conducted under soaked condition during monsoon season, there is no need to apply correction for subsequent soaking).

**California Bearing Ratio (CBR) test**

This is a penetration test developed by the California Division of Highways, as a method for evaluating the stability of soil subgrade and other flexible pavement materials. The test results have been correlated with flexible pavement thickness requirements for highways and air fields. The CBR test may be conducted in the laboratory on a prepared specimen in a mould or in-situ in the field.

The laboratory CBR apparatus consists of a mould 150 mm diameter with a base plate and a collar, a loading frame with the cylindrical plunger of 50 mm diameter and dial gauges for measuring the expansion on soaking and the penetration values, refer Fig. 6.13.

Briefly the penetration test consists of causing a cylindrical plunger of 50 mm diameter to penetrate a pavement component material at 1.25 mm/minute. The load values to cause 2.5 mm and 5.0 mm penetration are recorded. These loads are expressed as percentages of standard load values at respective deformation levels to obtain CBR value. The standard load values obtained from the average of a large number of tests on crushed stones are 1370 and 2055 kg (70 and 105 kg/cm$^2$) respectively at 2.5 and 5.0 mm penetration.

The specimen in the mould is subjected to four days soaking and the swelling and water absorption values are noted. The surcharge weight is placed on the top of the



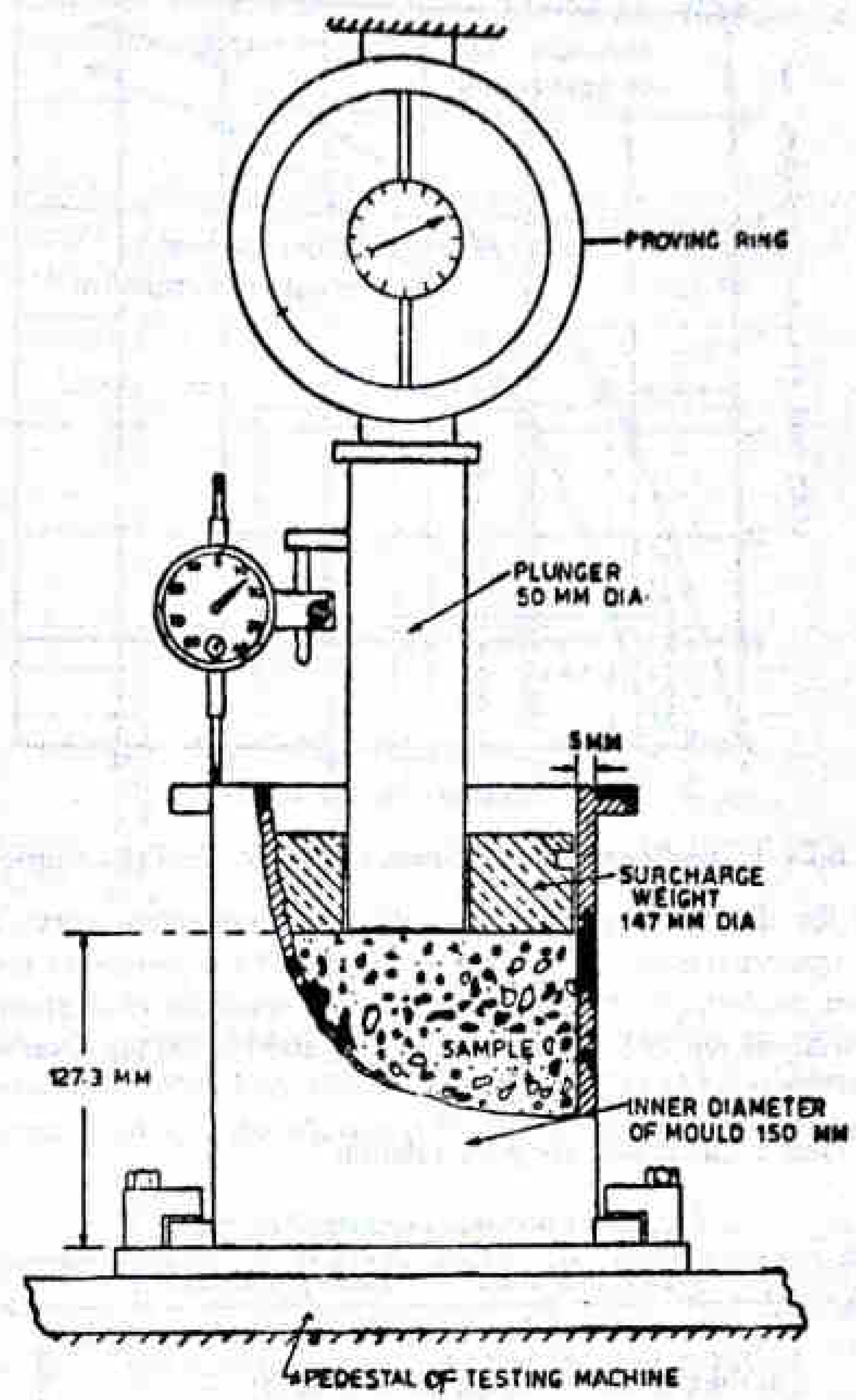



**Fig. 6.13 CBR Test Set up**

specimen in the mould and the assembly is placed under the plunger of the loading frame as shown in Fig. 6.13. The load values are noted corresponding to penetration values 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. The load penetration graph is plotted as shown in Fig. 6.14. Alternatively the load values may be converted to pressure values and plotted against the penetration values.

Two typical types of curves may be obtained as shown in Fig. 6.14. The normal curve is with convexity upwards as for Specimen no.1 and the loads corresponding to 2.5 and 5.0 mm penetration values are noted. Some times a curve with initial upward concavity is obtained, indicating the necessity of correction as for Specimen no. 2. In this case the corrected origin is established by drawing a tangent AC from the steepest point A on the curve. The load values corresponding to 2.5 and 5.0 mm penetration values from the corrected origin C are noted.

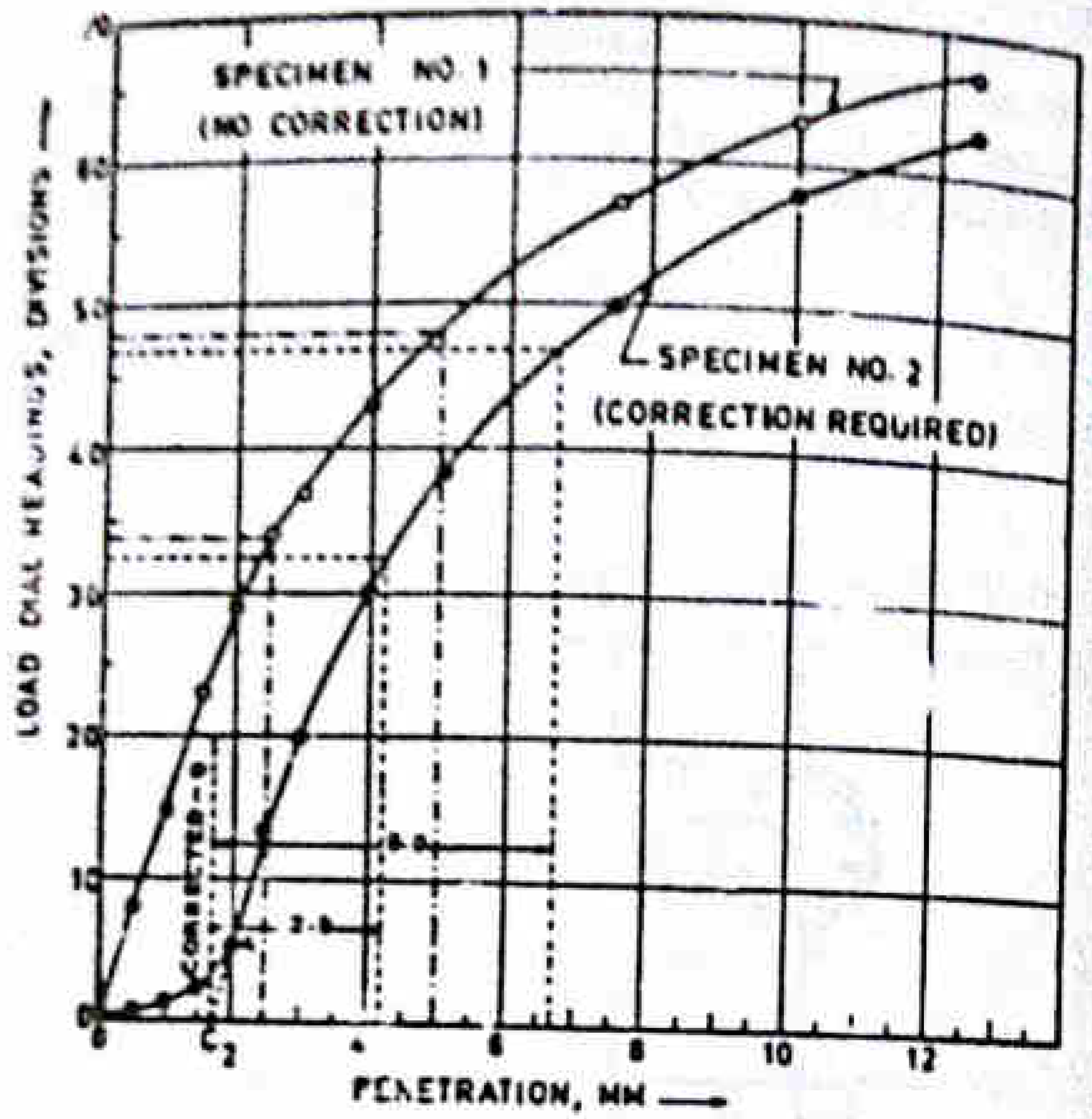



**Fig. 6.14 Load-Penetration Curves in C.B.R. Test (Example 6.4)**

The causes for the initial concavity of the load-penetration curve calling for the correction in origin are due to : (i) the bottom surface of the plunger or the top surface of the soil specimen not being truly horizontal, with the result the plunger surface not being in full contact with the top of the specimen initially and (ii) the top layer of the specimen being too soft or irregular.

The C.B.R. value is calculated using the relation :

$$\text{CBR, \%} = \frac{\left[\begin{array}{l}\text{Load (or pressure) sustained by the} \\ \text{specimen at 2.5 or 5.0 mm penetration}\end{array}\right]}{\left[\begin{array}{l}\text{Load (or pressure) sustained by standard aggregates} \\ \text{at the corresponding penetration level}\end{array}\right]} \times 100 \qquad (6.8)$$

Normally the CBR value at 2.5 mm penetration which is higher than that at 5.0 mm is reported as the CBR value of the material. However, if the CBR value obtained from the test at 5.0 mm penetration is higher than that at 2.5 mm, then the test is to be repeated for checking. If the check test again gives similar results, the higher value obtained at 5.0 mm penetration is reported as the CBR value. The average CBR value of three test specimens is reported to the first decimal place, as the CBR value of the material. If the variation in CBR value between the three specimens is more than the prescribed limits, tests should be repeated on additional three samples and the average CBR value of six specimens is accepted.

The CBR test is essentially an arbitrary strength test and hence can not be used to evaluate the soil properties like cohesion or angle of internal friction or shearing resistance. Unless the test procedure is strictly followed, dependable results cannot be obtained. Presence of coarse grained particles would result in poor reproducibility of results. Material passing 20 mm sieve is only used in the test. The field CBR test is carried out using in-situ penetration test.



The test is meant for soils and is also carried out on sub-base and granular base course materials. The CBR test values are made use of in an empirical method of flexible pavement design as given in Chapter 7.

**Example 6.4**

The load penetration values of CBR tests conducted on two specimens of a soil sample are given below. Determine the CBR value of the soil if 100 division of the load dial represents 190 kg load in the calibration chart of the proving ring.

| Penetration of plunger, mm | Load dial readings, divisions | |
|---|---|---|
| | Specimen No 1 | Specimen No 2 |
| 0.0 | 0 | 0 |
| 0.5 | 8 | 0.5 |
| 1.0 | 15 | 1.5 |
| 1.5 | 23 | 2.5 |
| 2.0 | 29 | 6.0 |
| 2.5 | 34 | 13 |
| 3.0 | 37 | 20 |
| 4.0 | 43 | 30 |
| 5.0 | 48 | 38 |
| 7.5 | 57 | 50 |
| 10.0 | 63 | 58 |
| 12.5 | 67 | 63 |

**Solution**

The penetration values are plotted against the load dial reading as shown in Fig. 6.14. (Instead, the load dial readings may also be converted either to load values in kg or load per unit area of cross section of the plunger in kg/cm$^2$ and plotted on the Y-axis.)

*Specimen no. 1*

The load penetration curve for specimen no.1 is consistently convex throughout and needs no correction.

Load dial reading at 2.5 mm penetration = 34 divisions (Fig. 6.14)

$$\text{Load at 2.5 mm penetration} = 34 \times \frac{190}{100} = 64.6 \text{ kg}$$

$$\text{CBR value at 2.5 mm penetration} = \frac{64.6 \times 100}{1370} = 4.7\%$$

$$\text{CBR value at 5.0 mm penetration} = \frac{48 \times 190 \times 100}{100 \times 2055} = 4.4\%$$

$$\therefore \text{CBR value of Specimen no.1} = 4.7\%$$

*Specimen no.2*

As the curve has an initial concavity, correction is required. A tangent AC is drawn from the steepest portion A of the curve to intersect the X-axis at C, which is the corrected origin for this specimen. The penetration values are measured from this corrected origin C, as shown in Fig. 6.14.

$$\text{CBR value at 2.5 mm penetration} = \frac{32.5 \times 190 \times 100}{100 \times 1370}$$

$$= 4.5\%$$

$$\text{CBR value at 5.0 mm penetration} = \frac{47 \times 190}{2055} = 4.3\%$$

$$\text{CBR value of Specimen no.2} = 4.5\%$$

$$\text{Therefore mean CBR value of the soil sample} = \frac{4.7 + 4.5}{2}$$

$$= 4.6\%$$

**Stabilometer Test**

Hveem's Stabilometer test is conducted on subgrade soil at various moisture contents. The stabilometer R-value is determined using Eq. 6.17 as explained at the end of this chapter. The result of this test are used in the Stabilometer method of pavement design (See Art. 7.3.4).

## 6.2 STONE AGGREGATES

### 6.2.1 Introduction

Aggregates form the major portion of pavement structure and they form the prime materials used in pavement construction. Aggregates have to bear stresses occurring due to the wheel loads on the pavement and on the surface course they also have to resist wear due to abrasive action of traffic. These are used in pavement construction in cement concrete, bituminous concrete and other bituminous constructions and also as granular base course underlying the superior pavement layers. Therefore the properties of the aggregates are of considerable significance to the highway engineers.

Most of the road aggregates are prepared from natural rock. *Gravel* aggregates are small rounded stones of different sizes which are generally obtained as such from some river beds. Sand is fine aggregate from weathering of rock. The properties of the rock from which the aggregates are formed, depend on the properties of constituent materials and the nature of bond between them. Based on the origin, natural rocks are classified as igneous, sedimentary and metamorphic. Texture is an important factor, affecting the property of the rock and the fragments.

The aggregates are specified based on their grain size, shape, texture and its gradation. Aggregate size is ascertained by sieving through square sieves of successively decreasing sizes. The required aggregate sizes are chosen to fulfil the desired gradation. The gradings for different road making purposes have been specified by various agencies like the A.S.T.M., B.S.I., I.S.I. and the I.R.C.

Based on the strength property, the coarse aggregates may be divided as *hard aggregates and soft aggregates*. Generally for the bearing course of superior pavement types, hard aggregates are preferred to resist the abrading and crushing effects of heavy traffic loads and to resist adverse weather conditions. In the case of low-cost road construction for use in lower layers of pavement structures, soft aggregates can also be used. The soft aggregate include *moorum, kankar, laterite, brick aggregates and slag*. A different set of test specifications are adopted for soft aggregates.



### 6.2.2 Desirable Properties of Road Aggregates

*Strength*

The aggregates to be used in road construction should be sufficiently strong to withstand the stresses due to traffic wheel load. The aggregates which are to be used in top layers of the pavements, particularly in the wearing course have to be capable of withstanding high stresses in addition to wear and tear; hence they should possess sufficient strength resistance to crushing.

*Hardness*

The aggregates used in the surface course are subjected to constant rubbing or *abrasion* due to moving traffic. They should be hard enough to resist the wear due to abrasive action of traffic. Abrasive action may be increased due to the presence of abrasive material like sand between the tyres of moving vehicles and the aggregates exposed at the top surface. This section may be severe in the case of steel tyred vehicles. Heavy wheel loads can also cause deformations on some types of pavement resulting in relative movement of aggregates and rubbing of aggregates with each other within the pavement layer. The mutual rubbing of stones is called *attrition*, which also may cause a little wear in the aggregates; however attrition will be negligible or absent in most of the pavement layers.

*Toughness*

Aggregates in the pavements are also subjected to impact due to moving wheel loads. Sever impact like hammering is quite common when heavily loaded steel tyred vehicles move on water bound macadam roads where stones protrude out especially after the monsoons. Jumping of the steel tyred wheels from one stone to another at different levels causes severe impact on the stones. The magnitude of impact would increase with the roughness of the load surface, the speed of the vehicle and other vehicular characteristics. The resistance to impact or toughness is hence another desirable property of aggregates.

*Durability*

The stone used in pavement construction should be durable and should resist disintegration due to the action of weather. The property of the stones to withstand the adverse action of weather may be called *soundness*. The aggregates are subjected to the physical and chemical action of rain and ground water, the impurities there-in and that of atmosphere. Hence it is desirable that the road stones used in the construction should be sound enough to withstand the weathering action.

*Shape of Aggregates*

The size of the aggregates is first qualified by the size of square sieve opening through which an aggregate may pass, and not by the shape. Aggregates which happen to fall in a particular size range may have rounded, cubical, angular flaky or elongated shape of particles. It is evident that the flaky and elongated particles will have less strength and durability when compared with cubical, angular or rounded particles of the same stone. Hence too flaky and too much elongated aggregates should be avoided as far as possible. Rounded aggregates may be preferred in cement concrete mix due to low specific surface area and better workability for the same proportion of cement paste and same water cement ratio, whereas rounded particles are not preferred in granular base course, WBM construction and bituminous construction as the stability due to interlocking of rounded particles is less. In such constructions angular particles are preferred. The voids present in a compacted mix of coarse, aggregates depends on the shape factors. Highly angular flaky and elongated aggregates have more voids in comparison with rounded aggregates.

*Adhesion with Bitumen*

The aggregates used in bituminous pavements should have less affinity with water when compared with bituminous materials; otherwise the bituminous coating on the aggregate will be stripped off in presence of water.

### 6.2.3 Tests for Road Aggregate

In order to decide the suitability of the road stones for use in construction, the following tests are carried out :

(a) Crushing test

(b) Abrasion test

(c) Impact test

(d) Soundness

(e) Shape test

(f) Specific gravity and water absorption test

(g) Bitumen adhesion test

The essential features of these tests are discussed below. Separate tests are available for testing cylindrical stone specimens and coarse aggregates for crushing, abrasion and impact tests. But due to the difficulties of preparing cylindrical stone specimen which need costly core drilling, cutting and polishing equipment, the use of such tests are now limited. Testing of aggregates is easy and simulate the field condition better, as such these are generally preferred.

**Aggregate crushing test**

The strength of coarse aggregate may be assessed by aggregate crushing test. The aggregate crushing value provides a relative measure of resistance to crushing under gradually applied compressive load. To achieve a high quality of pavement, aggregates possessing high resistance to crushing or low aggregate crushing value are preferred.

The apparatus for the standard test consists of a steel cylinder 15.2 cm diameter with a base plate and a plunger, compression testing machines, cylindrical measure of diameter 11.5 cm and height 18 cm, tamping rod and sieves. The sketch of the test cylinder and accessories is shown in Fig. 6.15.

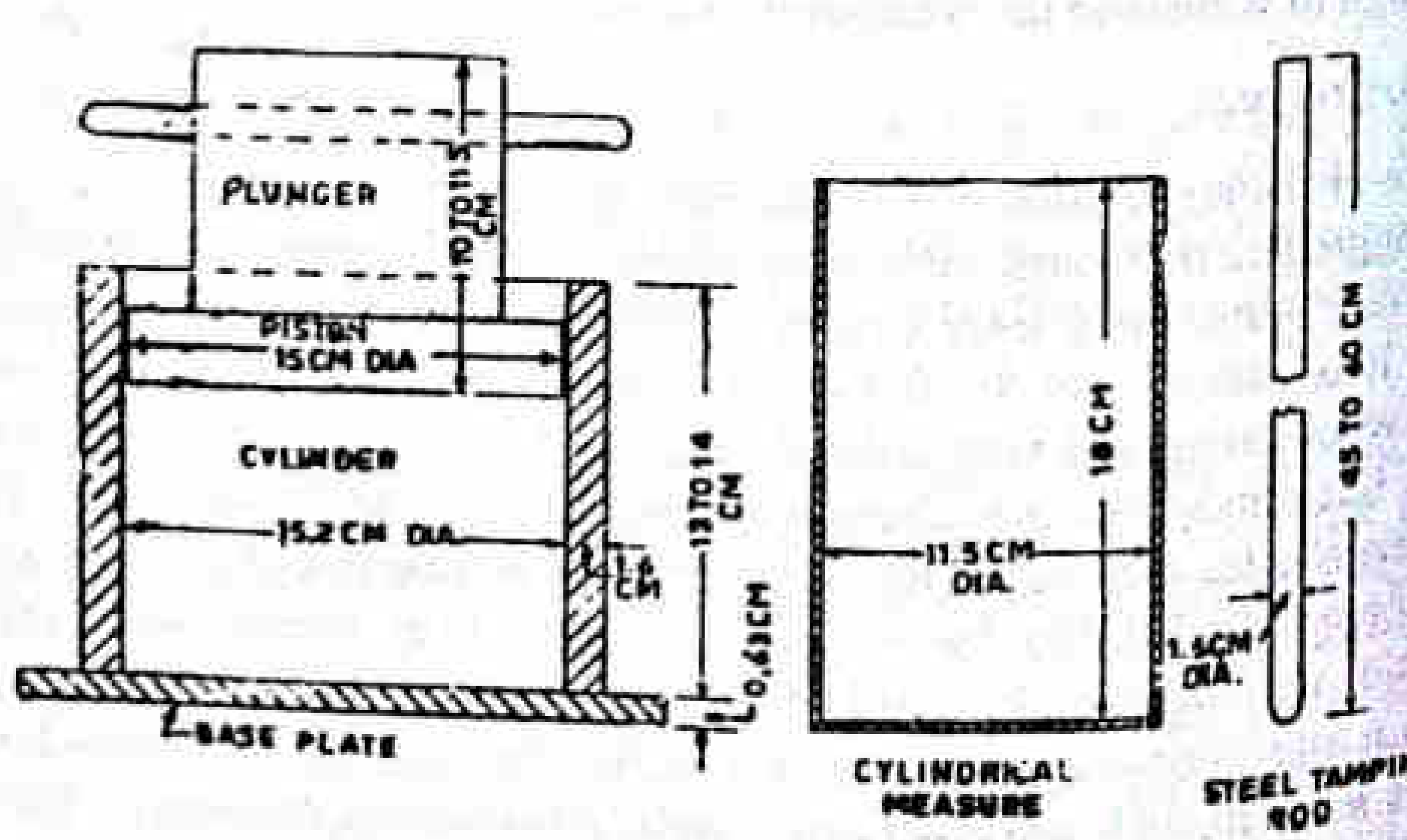



**Fig. 6.15 Aggregate Crushing Test Apparatus**



Dry aggregate passing 12.5 mm IS sieve and retained on 10 mm sieve is filled in the cylindrical measure in three equal layers, each layer being ramped 25 times by the tamper. The test sample is weighed (equal to $W_1$ g) and placed in the test cylinder in three equal layers, tamping each layer 25 times. The plunger is placed on the top of specimen and a load of 40 tonnes is applied at a rate of 4 tonnes per minute by the compression machine. The crushed aggregate is removed and sieved on 2.36 mm IS sieve. The crushed material which passes this sieve is weighed equal to $W_2$ g. The aggregate crushing value is the percentage of the crushed material passing 2.36 mm sieve in terms of original weight of the specimen.

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

Strong aggregates give low aggregate crushing value. The aggregate crushing value for good quality aggregate to be used in base course shall not exceed 45 percent and the value for surface course shall be less than 30 percent.

**Abrasion tests**

Due to the movements of traffic the road stones used in the surface course are subjected to wearing action at the top. Hence road stones should be hard enough to resist the abrasion due to the traffic. Abrasion tests are carried out to test the hardness property of stones and to decide whether they are suitable for the different road construction works. The abrasion test on aggregate may be carried out using any one of the following three tests :

(i) Los Angeles abrasion test

(ii) Deval abrasion test

(iii) Dorry abrasion test

However Los Angeles abrasion test is preferred as the test results have been correlated with pavement performance.

*Los Angeles Abrasion Test*

The principle of Los Angeles abrasion test is to find the percentage wear due to the relative rubbing action between the aggregate and steel balls used as abrasive charge. Pounding action of these balls also exists during the test and hence the resistance to wear and impact is evaluated by this test. The Los Angeles machine consists of a hollow cylinder closed at both ends, having inside diameter 70 cm and length 50 cm and mounted so as to rotate about its horizontal axis. The machine is shown in Fig. 6.16. The abrasive charge consists of cast iron spheres of approximate diameter 4.8 cm and each of weight 390 to 445 g. The number of spheres to be used as abrasive charge and their total weight have been specified based on grading of the aggregate sample. The test has been standardised by the ISI.

The specified weight of aggregate specimen, (5 to 10 kg, depending on gradation) is placed in the machine along with the abrasive charge. The machine is rotated at a speed of 30 to 33 rpm for the specified number of revolutions (500 to 1000 depending on the grading of the specimen). The abraded aggregate is then sieved on 1.7 mm IS sieve, and the weight of powdered aggregate passing this sieve is found. The result of the abrasion test expressed as the percentage wear or the percentage passing 1.7 mm sieve expressed

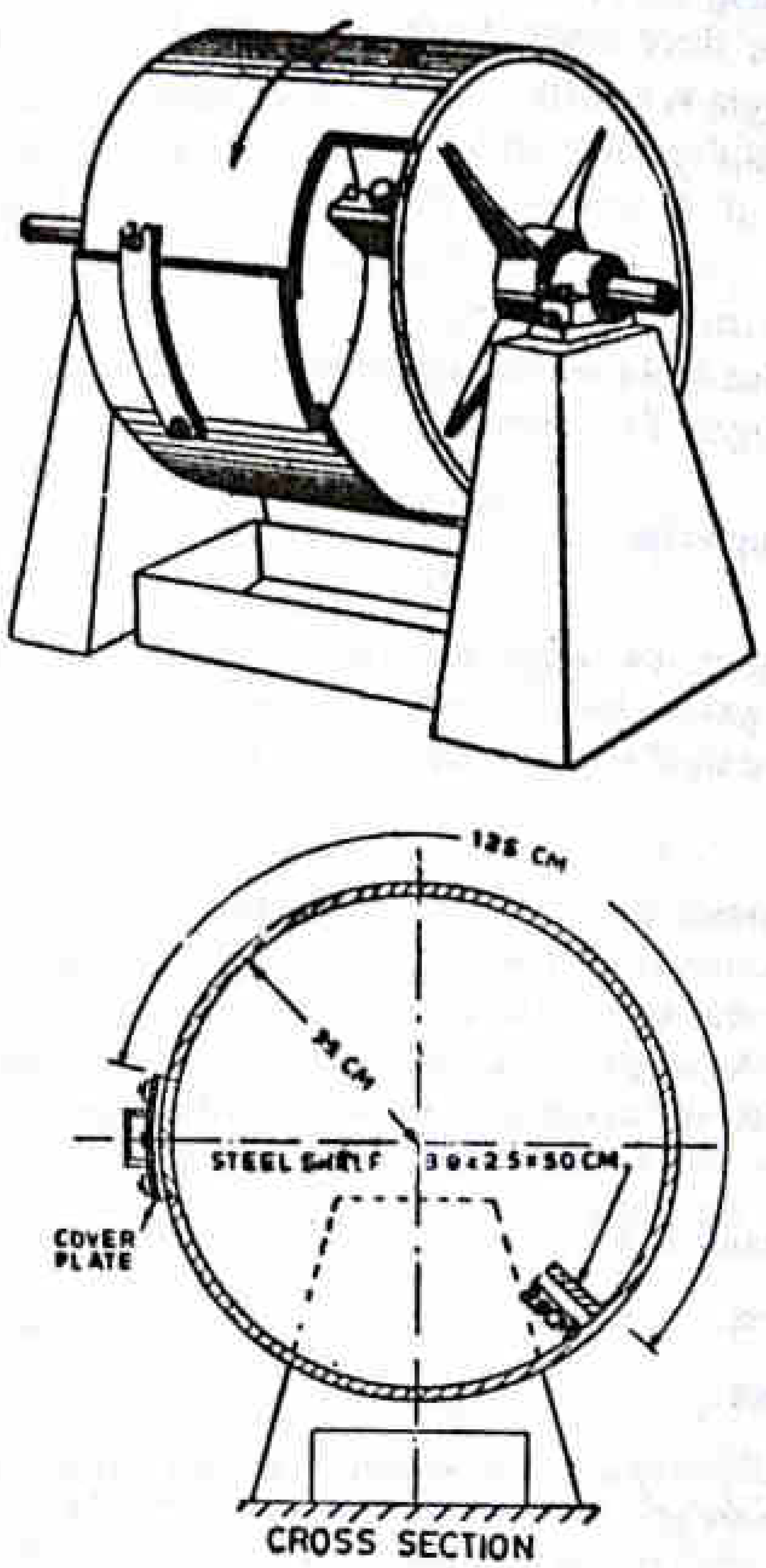



**Fig. 6.16 Los Angeles Abrasion Testing Machine**

in terms of the original weight of the sample. The Los Angeles abrasion value of good aggregates acceptable for cement concrete, bituminous concrete and other high quality pavement materials should be less than 30 percent. Values up to 50 percent are allowed in base courses like water bound and bituminous macadam. This test is more dependable than other abrasion tests as rubbing and pounding action in the test simulate the field conditions better. Also correlation of Los Angeles abrasion value with field performance and specifications of the test values have been established.

*Deval Abrasion Test*

The principle of the test is by allowing the sample of aggregate specimen to tumble over in a rattler in the presence of abrasive charge. The Devel machine consists of two hollow cylinder of diameter 20 cm and length 34 cm mounted in such a way that the cylinder rotate about a horizontal axis, but the axis of the cylinders make 30° angle with the horizontal. The schematic sketch of the machine is shown in Fig. 6.17. Specified quantity of dry aggregate specimen (4 to 5.5 kg), of any one of the specified gradings is placed in a cylinder. The abrasive charge consisting of 6 cast iron or steel spheres of



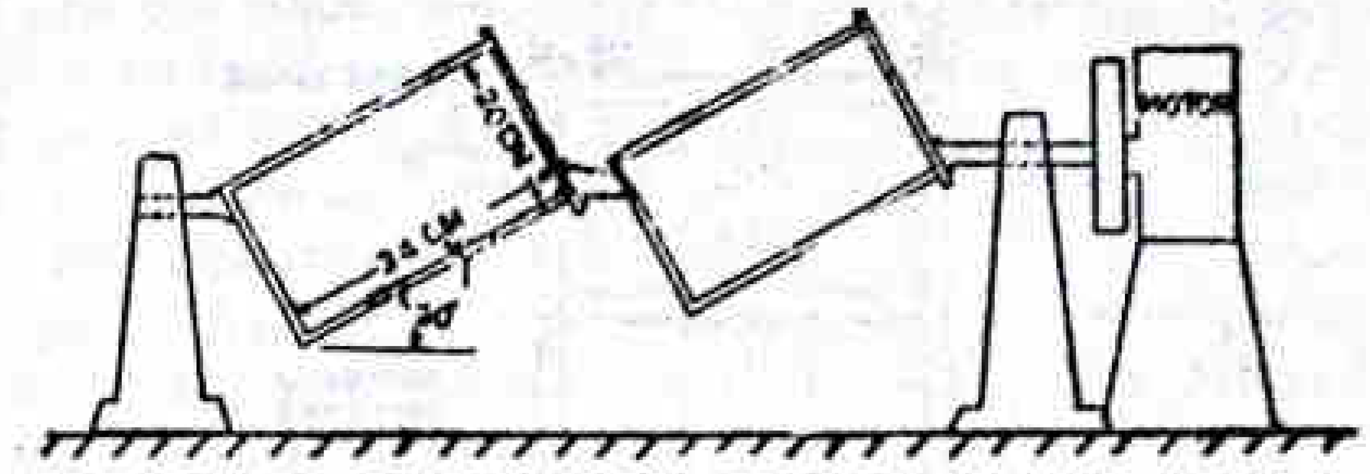



**Fig. 6.17 Deval Abrasion Testing Machine**

about 4.8 cm diameter and total weight 2500 g is placed. Two tests may be carried out simultaneously using both the cylinders. The machine is rotated at a speed of 30 to 33 rpm. After 10,000 revolutions the material is sieved on 1.7 mm IS sieve. The material passing this sieve is expressed as the percentage of the original weight of the sample and is reported as the abrasion value.

When the test is carried out by Deval machine without using abrasive charges, the test is known as Deval *attrition test*. However this test is not commonly carried out.

*Dorry Abrasion Test*

The abrasion value of aggregate is also determined using Dorry abrasion testing machine. This is a British method. The machine consists of a flat circular iron disc of 60 cm diameter which is rotated in a horizontal plane at 28 to 30 rpm. Two rectangular trays are kept 26 cm from the centre of the disc to hold the aggregate sample in a specified manner. Abrasive sand is fed through the funnel and the disc is subjected to 500 revolutions. The abrasion value is expressed as the percent loss in weight due to abrasion.

**Impact test**

A test designed to evaluate the toughness of stone or the resistance of the aggregates to fracture under repeated impacts is called impact test. The aggregate impact test is commonly carried out to evaluate the resistance to impact of aggregates and has been standardised by ISI.

The aggregate impact value indicates a relative measure of resistance of aggregate to impact, which has a different effect than the resistance to gradually increasing compressive stress. The aggregate impact testing machine consists of a metal base and a cylindrical steel cup of internal diameter 10.2 cm and depth 5 cm in which the aggregate specimen is placed. A metal hammer of weight of 13.5-14.0 kg having a free fall from a height 38 cm is arranged to drop through vertical guides. The aggregate impact machine is shown in Fig. 6.18.

Aggregate specimen passing 12.5 mm sieve and retained on 10 mm sieve is filled in the cylindrical measure in 3 layers by tamping each layer by 25 blows. The sample is transferred from the measure to the cup of the aggregate impact testing machine and compacted by tamping 25 times. The hammer is raised to a height of 38 cm above the upper surface of the aggregate in the cup and is allowed to fall freely on the specimen. After subjecting the test specimen to 15 blows, the crushed aggregate is sieved on 2.36 mm sieve. The aggregate impact value is expressed as the percentage of the fine formed in terms of the total weight of the sample.

The aggregate impact value should not normally exceed 30 percent for aggregate to be used in wearing course of pavements. The maximum permissible value is 35% for bituminous macadam and 40% for water bound macadam base courses.

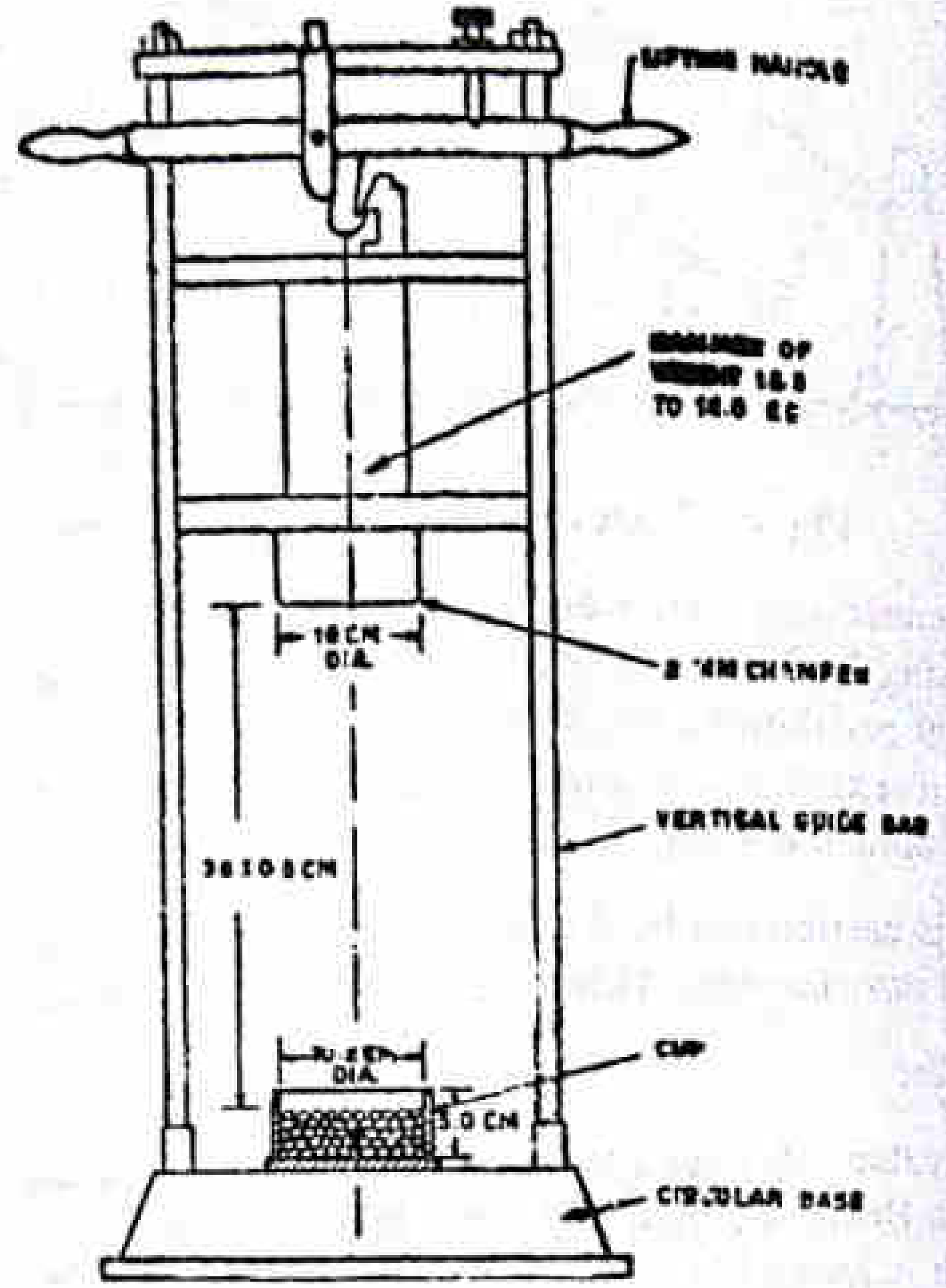



**Fig. 6.18 Aggregate Impact Testing Machine**

### Soundness test

Soundness test is intended to study the resistance of aggregates to weathering action, by conducting accelerated weathering test cycle. In order, to quicken the effects of weathering due to alternate wet-dry and/or freeze-thaw cycles in the laboratory, the resistance to disintegration of aggregate is determined by using saturated solution of *sodium sulphate* or *magnesium sulphate*. Clean, dry aggregate specimen of specified size range is weighed and counted. It is immersed in the saturated solution of sodium sulphate or magnesium sulphate for 16 to 18 hours. Then the specimen is dried in an oven at 105-110°C to a constant weight, thus making one cycle of immersion and drying. The number of such cycles is decided by prior agreement and then the specimens are tested. After completing the final cycle, the sample is dried and each fraction of the aggregate is examined visually to see if there is any evidence of excessive splitting, crumbling or disintegration of the grains. Sieve analysis is carried out to note the variation in gradation from the original. The coarse aggregate fraction of each size range is sieved on specified sieve sizes. The average loss in weight of aggregates to be used in pavement construction after 10 cycles should not exceed 12 percent when tested with sodium sulphate and 18 percent when tested with magnesium sulphate.

### Shape tests

The particle shape of aggregate mass is determine by the percentages of flaky and elongated particles contained in it and by its angularity. The evaluation of shape of the particles made in terms of flakiness index, elongation index and angularity number.

*Flakiness Index*

The flakiness index of aggregate is the percentage by weight of aggregate particles whose least dimension/thickness is less than three fifths or 0.6 of their mean dimension.



The test is applicable to sizes larger than 6.3 mm. Standard thickness gauge is used to gauge the thickness of the samples. The sample of aggregates to be tested is sieved through a set of sieves and separated into specified size ranges. Now to separate the flaky material, the aggregates which pass through the appropriate elongated slot of the thickness gauge are found. The width of the appropriate slot would be 0.6 of the average of the size range. If the size range of aggregate in a group is 16-20 mm, the width of the slot to be selected in thickness gauge would be $18 \times 0.6 = 10.8$ mm. The flaky material passing the appropriate slot from each size range of test aggregates are added up and let this weight be w. If the total weight of sample taken from the different size ranges is W, the flakiness index is given by 100 w/W percent, or in other words it is the percentage of flaky materials, the widths of which are less than 0.6 of the mean dimensions. It is desirable that the flakiness index of aggregates used in road construction is less than the 15 percent and normally does not exceed 25 percent.

*Elongation Index*

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

The sample of aggregate to be tested is sieved through a set f sieve and separated into specified size ranges. The aggregates from each of the size range is then individually passed through the appropriate gauge of the length gauge with the longest side in order to separate the elongated particles. The gauge length would be 1.8 times the mean size of the aggregate. The portion of the elongated aggregate having length greater than the specified gauge from each range is weighed and the total weight of the elongated stones. is expressed as a percentage of the total weight of the sample, to get the elongation index.

Elongated and flaky aggregates are less workable; they are also likely to break under smaller loads than the aggregate which are spherical or cubical. Flakiness index and elongation index values in excess of 15 percent are generally considered undesirable; however no recognised limits have been laid down for elongation index.

*Angularity Number*

Based on the shape of the aggregate particles, they may be classified as rounded, irregular or partly 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 this test. Angularity or absence of 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. 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.

The degree of packing of particles of single sized aggregates depends on the shape and angularity of the aggregate. Hence the angularity of the aggregate can be estimated from the properties of voids in a sample of aggregate compacted in a particular manner. Angularity number is defined as 67 - percent solid volume. The solid volume of the aggregate is found by filling it in a vessel in a specified manner. In the expression for angularity number, the value 67 represents the volume of solids (in percent) of most rounded gravel in a well compacted state which would then have 33 percent voids. Thus the angularity number measures the voids in excess of 33 percent. The higher the number, more angular is the aggregate. The range of angularity number for aggregates used in constructions is 0 to 11.

The apparatus for testing the angularity number consists of a metal cylinder of capacity 3 litre, tamping rod and a metal scoop. The test sample is sieved and a specified size ranges of the aggregate, such as 16 – 20 mm, 12.5 – 16 mm, etc. are used for the test. A scoop full of this single size aggregate is placed in the cylinder and tamped 100 times by the rod. Second and third layers are placed and tamped similarly and the excess aggregate is struck off level to the top surface of the cylinder. The weight of aggregate in the cylinder is found to be Wg. Then the cylinder is found = Cg. The specific gravity Ga of the aggregate is also determined. The angularity number is found from the formula:

$$\text{Angularity number} = 67 - \frac{100\ W}{CGa} \qquad (6.9)$$

This value is expressed as the nearest whole number.

**Specific gravity and water absorption tests**

The specific gravity of an aggregate is considered to a measure of the quality or strength of the material. Stones having low specific gravity values are generally weaker than those having higher values. The specific gravity test also helps identifying the stone specimen. Stones having higher water absorption value are porous and thus weak. They are generally unsuitable unless found acceptable based on crushing and hardness tests.

About 2 kg of dry aggregate sample is placed in wire basket and immersed in water for 24 hours. The sample is weighed in water and the buoyant weight is found. The aggregates are then taken out weighed after drying the surface. Then the aggregates are dried in an oven for 24 hours at a temperature 100 - 110°C, and then the dry weight is determined. The specific gravity is calculated by dividing the dry weight of aggregate by weight of equal volume of water. The water absorption is expressed as the percent water absorbed in terms of over dried weight of the aggregates.

The specific gravity of rocks very from 2.6 to 2.9. Rock specimens having more than 0.6 percent water absorption are considered unsatisfactory unless found acceptable based on strength tests. However slightly higher value of porosity may be acceptable for aggregates used in bituminous pavement construction, if the aggregates are found otherwise suitable.

**Bitumen adhesion test**

Bitumen and tar adhere well to all normal types of road aggregates provided they are dry and are free from dust. The process of initial binding is controlled largely by the viscosity of the binder. In the absence of water there is practically no adhesion problem in bituminous construction. The 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 binder from coated aggregate due to presence of water. This problem of *stripping* is generally experienced only with bituminous mixtures which are permeable to water. The stripping is due to the fact that some aggregates have greater 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 rocks possess a weak negative charge and hence these have greater attraction with the polar liquid water than with bituminous binders having



little polar activity. These aggregates which are electronegative are water-linking and are called *hydrophillic*. Basic aggregates like lime-stones have a dislike for water and greater attraction to bitumen, as they have positive surface charge. These aggregates are called *hydrophobic*.

It is important to know the type of charge of aggregates used in bituminous construction. Now bitumen is also available as cationic or positive and anionic or negative and hence a suitable selection may be made depending on aggregates available. Cationic (+) bitumen may be selected for electronegative aggregate and anionic (–) bitumen for electropositive aggregates.

Several laboratory tests have been developed to arbitrarily determine the adhesion of bituminous binder to an aggregate in presence of water. These tests may be classified into six types.

(i) Static immersion test

(ii) Dynamic immersion test

(iii) Chemical immersion test

(iv) Immersion mechanical test

(v) Immersion trafficking test and

(vi) Coating test

The static immersion test is very commonly used as it is quite easy and simple. The principle of this type of test is by immersing aggregate fully coated with the binder in water maintained at specified temperature and by estimating the degree of stripping. The result is reported as the percentage of stone surface that is stripped off after the specified time periods. IRC has specified that stripping value of aggregates should not exceed 25 percent for use in bituminous surface dressing, penetration macadam, bituminous macadam and carpet constructions, when aggregate coated with bitumen is immersed in water bath at 40°C for 24 hours.

## 6.3 BITUMINOUS MATERIALS

### 6.3.1 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, black in colour though they have different characteristics. Both these materials can be used for pavement works.

Bitumen is hydrocarbon material of either natural or pyrogenous origin, found in gaseous, liquid, semisolid or solid form and is completely soluble in *Carbon disulphide* and in *carbon tetra chloride*. Bitumen is a complex organic material and occurs either naturally or may be obtained artificially during the distillation of petroleum. Bituminous materials are very commonly used in highway construction because of their binding and their water proofing properties.

When the bitumen contains some inert material or *minerals*, it is some times called asphalt. Asphalt is found as deposits in the form of natural asphalt or rock asphalt.

The grades of bitumen used for pavement construction work of roads and airfields are called paving grades and those used for water proofing of structures and industrial floors etc. are called industrial grades. The paving bitumen available in India are classified into two categories :

(i) paving bitumen from Assam petroleum, denoted as A-type and designated as grades A35, A 90, etc.

(ii) paving bitumen from other sources denoted as S-type and designated as grades S 35, S 90, etc.

### 6.3.2 Types of Bituminous Materials

Bituminous material used in highway construction may be broadly divided as :

(i) Bitumen and

(ii) Tar

Bitumen may be further divided as petroleum asphalt or bitumen and native asphalt.

There are different forms in which native asphalts are available. Native asphalts are those which occur in a pure or nearly pure state in nature. Native asphalts which are associated with a large proportion of mineral matter are called rock asphalts. The viscosity of bitumen is reduced some times by a volatile diluent; this material is called *cutback*. When bitumen is suspended in a finely divided condition in an aqueous medium and stabilized with an emulsifier, the material is known as *emulsion*. Tar is the viscous liquid obtained when natural organic materials such as wood and coal are carbonized or destructively distilled in the absence of air. Processing of bitumen and bituminous products is diagrammatically represented in Fig. 6.19.

### 6.3.3 Bitumen

Crude petroleum obtained from different places are quite different in their composition. The portion of bituminous material present in the petroleums may widely differ depending on the source. Almost all the crude petroleums contain considerable amounts of water along with crude oil. Hence the petroleum should be dehydrated first before carrying out the distillation. General types of distillation processes are fractional distillation and destructive distillation. In fractional distillation the various volatile constituents are separated at successively higher temperatures without substantial chemical change. The successive fractions obtained yield gasoline, naptha, kerosene and lubricating oil; the residue would be petroleum bitumen. In destructive distillation the material undergoes chemical change under the application of extreme heat and pressure. The process is usually applied for the manufacture of tar. Steam distillation of petroleum is employed to produce steam refine petroleum bitumen in order to remove high boiling point constituents such as heavy lubricating oils without causing chemical changes. When the residue is distilled to a definite consistency without further treatment, the bitumen obtained as residue is called *straightrun bitumen*.

*Requirements of Bitumen*

The desirable properties of bitumen depend on the mix type and the construction. The general problems while using bitumen in paving mixes are :

(i) mixing



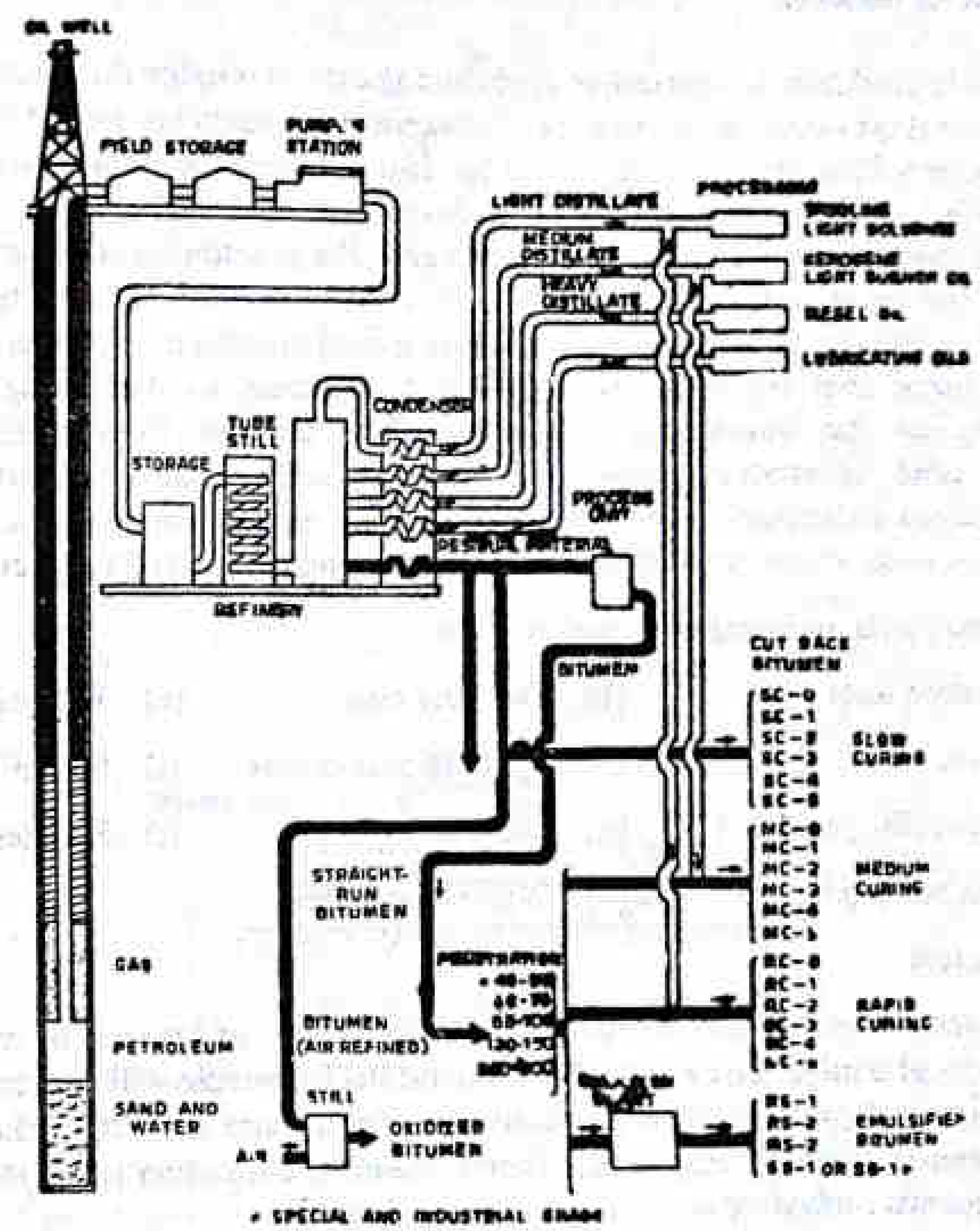



Fig. 6.19 Processing of Bituminous Products

(ii) attainment of desired stability of the mix

(iii) to maintain the stability under adverse weather conditions

(iv) to maintain sufficient flexibility and thus avoid cracking of bituminous surface and

(v) to have sufficient adhesion with the aggregates in the mix in presence of water.

In view of the above problems, the bitumen should possess the following desirable properties :

(i) The viscosity of the bitumen at the time of mixing and compaction should be adequate. This is achieved by heating the bitumen and aggregate prior to mixing or by use of cutbacks or emulsions of suitable grade.

(ii) The bituminous material should not be highly temperature susceptible. During the hottest weather of the region the bituminous mix should not become too soft or unstable. During cold weather the mix should not become too hard and brittle, causing cracking of surface. The material should be durable.

(iii) In presence of water the bitumen should not strip off from the aggregate. There has to be adequate affinity and adhesion between the bitumen and aggregate used in the mix.

### 6.3.4 Tests on Bitumen

Bitumen is available in a variety of types and grades. To judge the suitability of these binders various physical tests have been specified by agencies like ASTM, Asphalt Institute, British Standards Institution and the ISI. These tests include penetration test, ductility tests, softening point test and viscosity test. For classifying bitumen and studying the performance of bituminous pavements, the penetration and ductility tests are essential. The other tests like softening point and flash and fire point tests are more important to guide the paving technologists during field operations. In recent years, it has been recognized that the above tests are not sufficient to define the temperature susceptibility of the bituminous materials. The bitumen from different sources possessing same penetration value at a specified temperature may exhibit entirely different viscous characteristics at the application or service temperatures. These tests therefore may need intensive correlation with fundamental property like viscosity.

The various tests on bituminous materials are :

(a) Penetration tests
(b) Ductility tests
(c) Viscosity tests
(d) Float test
(e) Specific gravity test
(f) Softening point test
(g) Flash and Fire point test
(h) Solubility test
(i) Spot test
(j) Loss on heating test
(k) Water content test

**Penetration test**

The penetration test determines the hardness or softness of bitumen by measuring the depth in tenths of a millimetre to which a standard loaded needle will penetrate vertically in five seconds. The sample is maintained at a temperature of 25°C. The concept of penetration test is shown in Fig. 6.20. Indian Standard Institution has standardized the equipment and test procedure.

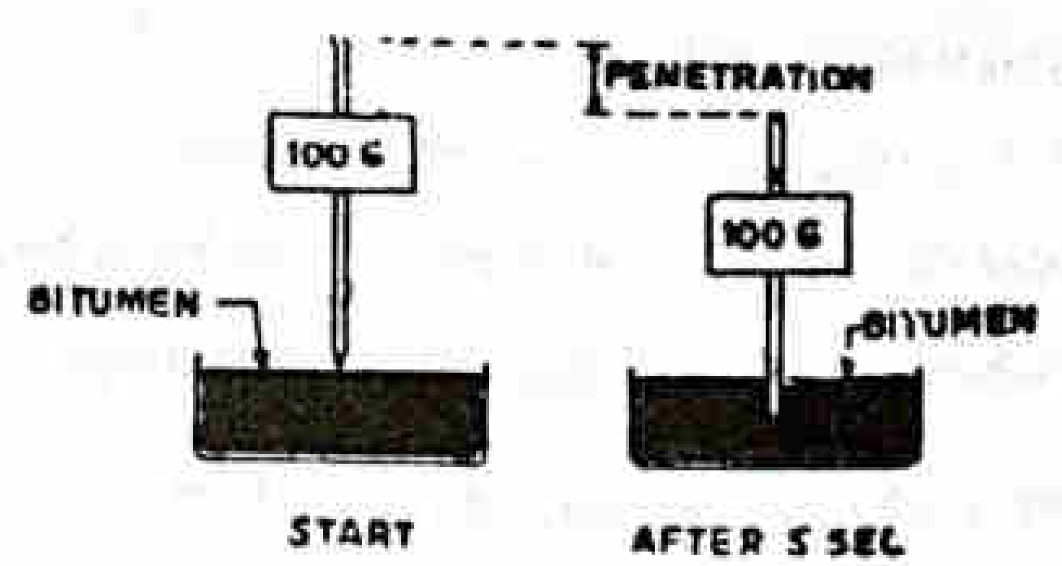



**Fig. 6.20 Penetration Test Concept**

The penetrometer consists of a needle assembly with a total weight of 100 g and device for releasing and locking in any position. There is a graduated dial to read penetration values to 1/10th of a millimeter. Refer Figure 6.21.

The bitumen is softened to a pouring consistency, stirred thoroughly and poured into containers to a depth at least 15 mm in excess of the expected penetration. The sample containers are then placed in a temperature controlled water bath at a temperature of 25°C for one hour. The sample with container is taken out and the needle is arranged to make contact with the surface of the sample. The dial is set to zero or the initial reading is



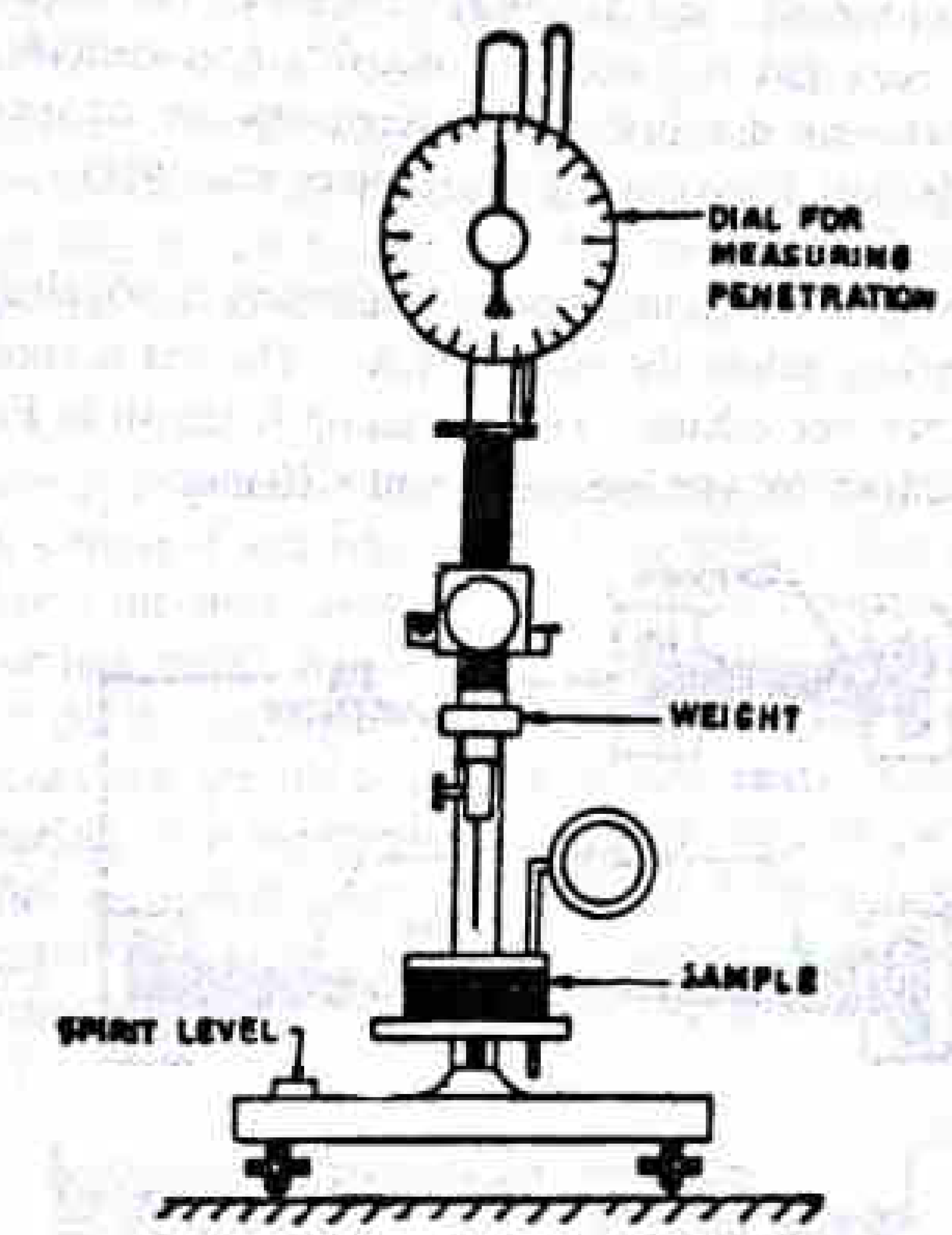



**Fig. 6.21 Penetrometer**

taken and the needle is released for 5 seconds. The final reading is taken on dial gauge. At least three penetration tests are made on this sample by testing at distances of atleast 10 mm apart. After each test the needle is disengaged and wiped with benzene and dried. The depth of penetration is reported in one-tenth millimeter units. The mean value of three measurements is reported as a penetration value. It may be noted that the penetration value is largely influenced by any inaccuracy as regards pouring temperature, size of needle, weight placed on the needle and the test temperature.

The bitumen grade is specified in terms of penetration value. 80-100 or 80/100 grade bitumen means that the penetration value of the bitumen is in the range 80 to 100 at standard test conditions. The penetration test is applied almost exclusively to bitumen. As road tars are soft, the penetration test cannot be carried out on these materials. Other consistency tests are used for tars, cutbacks and emulsions.

The penetration values of various types of bitumen used in pavement construction in this country range between 20 and 225. 30/40 and 80/100 grade bitumen are more commonly used, depending on construction type and climatic conditions. In hot climates a lower penetration grade bitumen like 30/40 bitumen is preferred.

**Ductility test**

In the flexible pavement constructions where bitumen binders are used, it is important that the binders form ductile thin films around the aggregates. This serves as a satisfactory binder in improving the physical interlocking of the aggregate bitumen mixes. Under traffic loads the bituminous pavement layer is subjected to repeated deformation and recoveries. The binder material which does not possess sufficient ductility would crack and thus provide pervious pavement surface. Ductility test is carried out on bitumen to test this property of the binder. The test is believed to measure the adhesive property of bitumen and its ability to stretch. 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 and ductility tests cannot 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°C and at a rate of pull of 50 mm per minute. The test set up is shown in Fig. 6.22. The cross section at minimum width of the specimen is 10 mm × 10 mm.

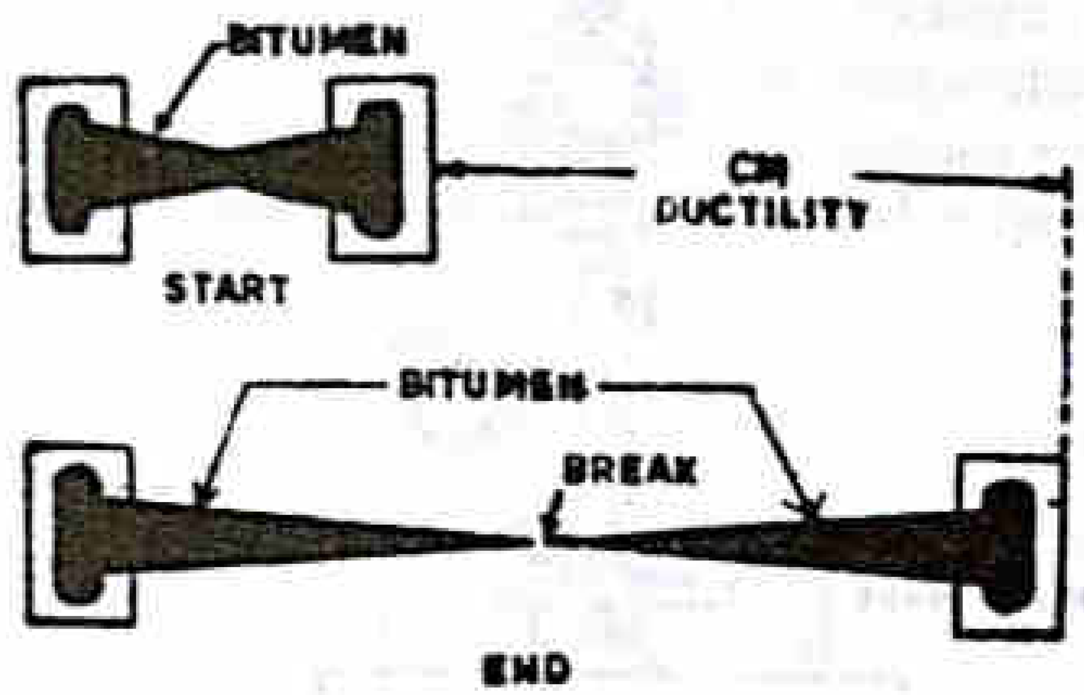



**Fig. 6.22 Ductility Test**

The ductility machine functions as a constant temperature water bath with a pulling device at a pre-calibrated rate. Two clips are thus pulled apart horizontally at a uniform speed of 50 mm per minute.

The bitumen sample is heated and poured in the mould assembly placed on a plate. The samples along with the moulds are cooled in air and then in water bath maintained at 27°C. The excess bitumen material is cut and the surface is leveled using a hot knife. The mould assembly containing sample is replaced in water bath of the ductility testing machine for 85 to 95 minute. The sides of the mould are removed, the clips hooked on the machine and the pointer is adjusted to zero. The distance upto the point of breaking of thread is reported in centimeters as ductility value. The ductility value gets seriously affected by factors such as pouring temperature, dimensions of briquette, level of briquette in the water bath, presence of air pockets in the modulus briquettes, test temperature and rate of pulling.

The ductility values of bitumen vary from 5 to over 100 for different bitumen grades. A minimum ductility value of 75 cm has been specified by the ISI for bitumens of grades 45 and above, obtained from sources other than Assam Petroleum (i.e., S 45, and above), the minimum ductility value may be 50 cm for bitumens of grades S 35, obtained from these sources. However, as the bitumen produced from Assam Petroleum in India have much lower ductility values, the minimum ductility value specified is only 15 cm for the bitumen grades A 65 to 200 for use in certain regions.

### Viscosity test

Viscosity is defined as inverse of fluidity. Viscosity thus defines the fluid property of bituminous material. Viscosity is the general term for consistency and it is measure of resistance to flow. Many researchers believe that grading of bitumen should be by absolute viscosity units instead of the conventional penetration units.



The degree of fluidity of the binder at the application temperature greatly influences the strength characteristics of the resulting paving mixes. High or low viscosity during mixing or compaction has been observed to result in lower stability values. There is an optimum value of viscosity for each aggregate gradation of the mix and bitumen grade. At low viscosity, the bituminous binder simply *lubricates* the aggregate particles instead of providing a uniform film for binding action. Similarly high viscosity also resists the compactive effort and the resulting mix is heterogeneous in character exhibiting low stability values.

Orifice type viscometer may be used to indirectly find the viscosity of liquid binders like cutback bitumen, emulsion and liquid tar. According to this method, viscosity is measured by determining the time taken by 50 ml of the material to flow from a cup through a specified orifice under standard test conditions and specified temperature. Higher the viscosity of the binder, higher will be the time required. This is illustrated in Fig. 6.23. Furol viscosity is a specific test which is used only to measure the viscosity of liquid bituminous materials. It is the number of seconds required for 50 ml of material to flow through an orifice of specified size at specified temperatures. Equipment like sliding plate microviscometer, and Brookfield viscometer are however in use for defining the viscous characteristics of the bitumen of all grades irrespective of testing temperature.

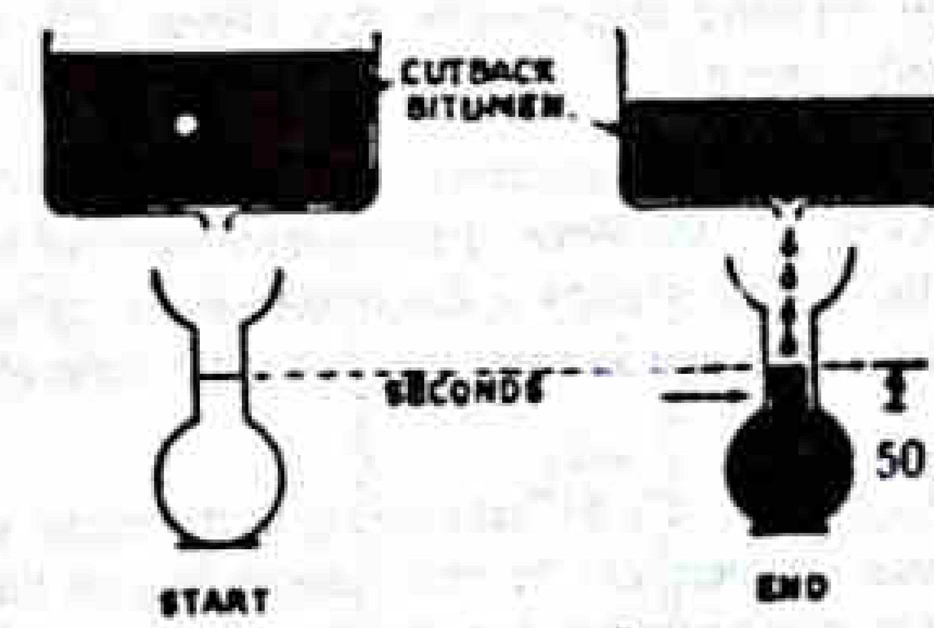



**Fig. 6.23 Viscosity Test set-up**

The viscosity of tar is determined as the time taken in seconds for 50 ml of the sample to flow through 10 mm orifice of the standard tar viscometer at the specified temperature of 35, 40, 45 or 55°C. The viscosity of cutback bitumen is determined as the time taken in seconds for 50 ml of the sample to flow through either 4.0 mm orifice at 25°C or 10 mm orifice at 25 or 40°C. Thus the orifice viscometer is suitable to test tars and cutbacks.

### Float test

There is a range 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 materials of this group is measured by float test.

The apparatus consists of a float made of aluminum and a brass collar filled with the specimen materials 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 into the float. Refer Fig. 6.24. The float assembly is floated in a water bath 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 higher the float test value, the stiffer is the material.

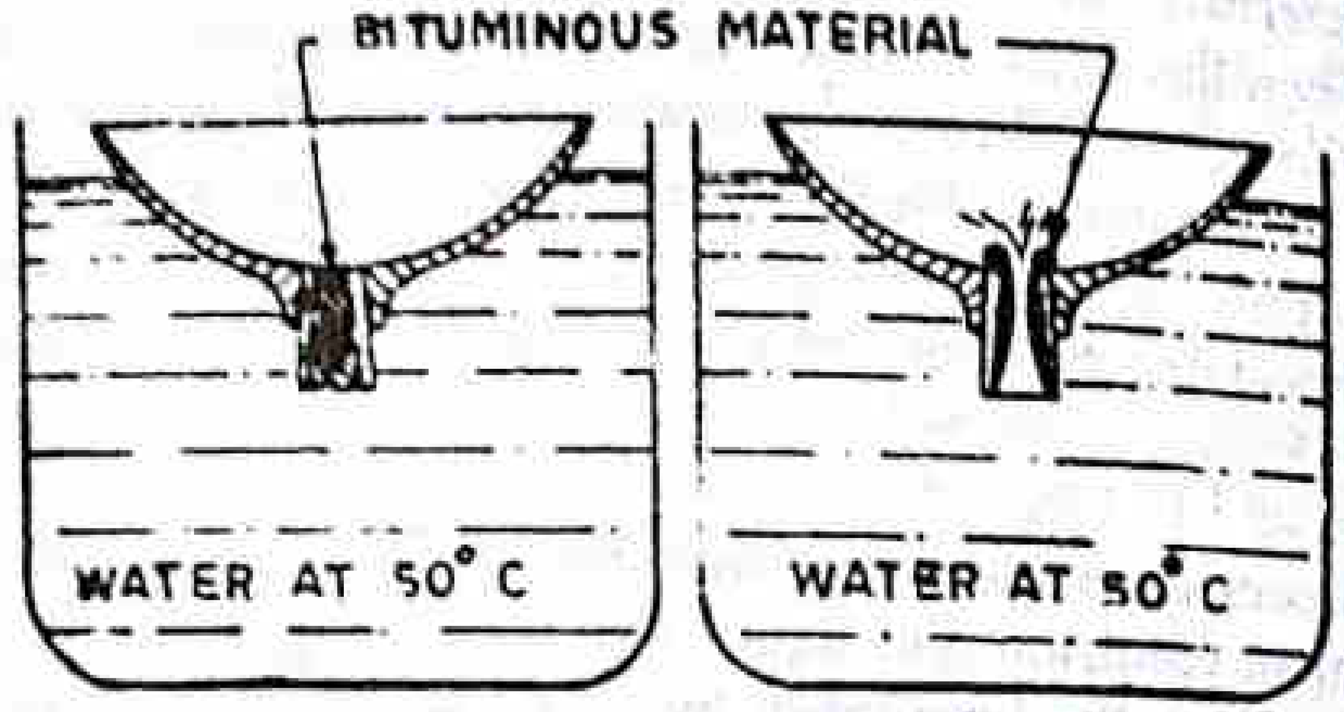



Fig. 6.24 Float Test Set-up

### Specific gravity test

The density of a bitumen binder is a fundamental property frequently used as an aid to classify the binders for use in paving jobs. In most applications, the bitumen is weighed, but finally when used with aggregate system, the bitumen content is converted on volume basis using density values. The specific gravity value of bitumen is also useful in bituminous mix design. The density of bitumen is greatly influenced by its chemical composition. Increased amounts of *aromatic type* compounds or mineral impurities cause an increase in specific gravity.

The specific gravity of bituminous materials is defined as the ratio of the mass of a given volume of the substance to the same of an equal volume of water, the temperature of both being 27°C. The specific gravity is determined either by using a pycnometer or by preparing a cube shape specimen in semi solid or solid state and by weighing in air and water.

Generally the specific gravity of pure bitumen is in the range of 0.97 to 1.02. The specific gravity of cutback bitumen may be lower depending on the type and proportion of diluent used. Tars have specific gravity ranging from 1.10 to 1.25.

### Softening point test

The softening point is the temperature at which the substance attains a particular degree of softening under specified condition of test. The softening point of bitumen is usually determined by Ring and Ball test. The test set-up is shown in Fig. 6.25.

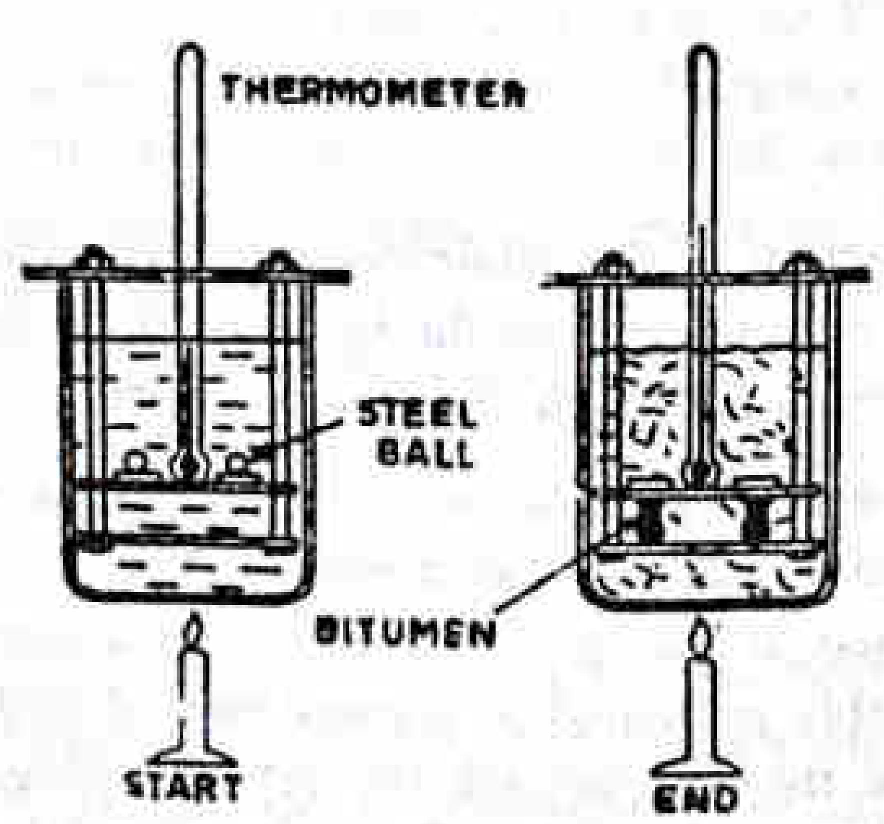



Fig. 6.25 Softening Point Test Set-up



Generally higher softening point indicates lower temperature susceptibility and is preferred in warm climates. A brass ring containing test sample of bitumen is suspended in liquid like water or glycerine at a given temperature. A steel ball is placed upon the bitumen sample and the liquid medium is then heated at a rate of 5°C per minute. The temperature at which the softened bitumen touches the metal placed at a specified distance below the ring is recorded as the softening point of a bitumen. Hard grade bitumen possess higher softening point than soft grade bitumens.

The softening point of various bitumen grades used in paving jobs vary between 35° to 70°C.

### Flash and fire point test

Bitumen materials leave out volatiles at temperatures depending upon their grade. These volatiles 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 paving engineers may restrict the mixing and application temperatures. As mentioned above, this test gives an indication of the critical temperature at and above which suitable precaution should be taken to eliminate fire hazards during heating of bitumen. The definition of flash and fire points as given by the ISI are as follows :

*Flash point* "The flash point of a material is the lowest temperature at which the vapour of a 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 conditions of test". Refer Fig. 6.26 for the test set up.

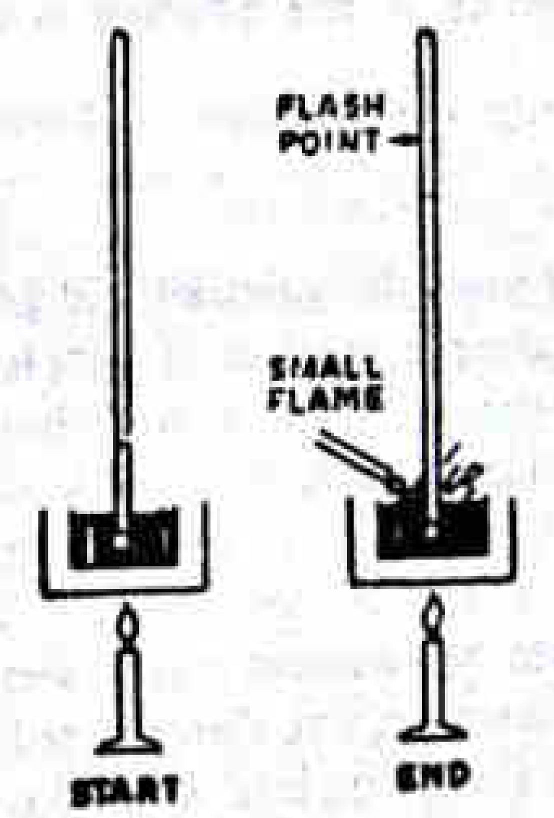



Fig. 6.26 Flash Point Test Set-up

*Pensky-Martens closed cup* apparatus or open cup are used for conducting the tests. The material to be tested 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 at the rate of 5° to 6°C per minute, stirring the specimen. 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 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 appear first at any point on the surface of the material. The heating is continued until the material gets ignited and continues to burn for 5 seconds; this temperature is recorded as the fire point.

The minimum specified flash point of bitumen used in pavement construction in Pensky Martens closed type test is 175°C.

**Solubility test**

Pure bitumen is completely soluble in solvents like carbon *disulphide* and *carbon tetrachloride*. Hence any impurity in bitumen in the form of inert minerals, carbon, salts etc. could be quantitatively analysed by dissolving the samples of bitumen in any of the two solvents. A sample of about 2 g of bitumen is dissolved in about 100 ml of solvent. The solution is filtered and the insoluble material retained is washed, dried and weighed; it is expressed as a percentage of original sample. The insoluble material should be preferably less than 1.0 percent. In solubility test with carbon tetrachloride, if black carbonaceous residue is over 0.5 percent, the bitumen is considered to be *cracked*. The minimum proportion of bitumen soluble in carbon disulphide is specified as 99 percent.

**Spot test**

This is a test for detecting over heated or *cracked* bitumen. This test is considered to be more sensitive than the solubility test for detection of cracking. About 2 g of bitumen is dissolved in 10 ml of *naphtha*. A drop of this solution is taken out and placed on a filter paper, one after one hour and second after 24 hours after the solution is prepared. If the strain of the spot on the paper is uniform in colour, the bitumen is accepted as uncracked. But if the spots form dark brown or black circle in the centre with an annular ring of lighter colour surrounding it, the bitumen is considered to be over heated or cracked.

**Loss on heating test**

When bitumen is heated, it loses the volatiles and gets hardened. To study the effect of heating, an accelerated heating procedure is adopted. About 50g of the sample is weighed and heated to a temperature of 163°C for 5 hours in a special oven designed for this test. This specimen is weighed again after the heating period and the loss in weight is expressed as a percentage by weight of original sample. Bitumen used in pavement mixes should not indicate more than one percent loss in weight; for bitumen of penetration values 150-200 upto two percent loss in weight is allowed. The residue after heating when subjected to penetration test shows a reduction in penetration value. The reduction in penetration value should be less than 40 percent of the original penetration value of the bitumen.

**Water content test**

It is desirable that the bitumen contains minimum water content to prevent foaming of the bitumen when it is heated above the boiling point of water. The water content in a bitumen is determined by mixing known weight of the specimen in a pure petroleum distillate free from water, heating and distilling off the water. The weight of the water condensed and collected is expressed as percentage by weight of the original sample. The maximum water content in bitumen should not exceed 0.2 percent by weight.

Explanations for some of the above tests have been based on the publication of *Burmah Shell* (See list of references).



### 6.3.5 Cutback Bitumen

Cutback bitumen is defined as the bitumen, the viscosity of which has been reduced by a volatile diluent. For use in surface dressings, some type of bitumen macadam and soilbitumen stabilization, it is necessary to have a fluid binder which can be mixed relatively at low temperatures. Hence to increase fluidity 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 cutback develops the binding properties. The viscosity of the cutback and rate of which it hardens on the road depend on the characteristics and quantity of both 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. The grade of cutback or its fluidity is designed by a figure which follows the initials; as an example RC-2 means that it is a rapid curing cutback of grade 2.

The cutback with the lowest viscosity is designated by numeral 0, such as RC-0, MC-0 and SC-0. Suffix numerals 0, 1, 2, 3, 4 and 5 designate progressively thicker or more viscous cutbacks as the numbers increase. This number indicates a definite viscosity irrespective of the type of cutback; in other words, RC-2, MC-2 and SC-2 all have the same initial viscosity at a specified temperature. The initial viscosity values (in seconds, standard tar viscometer) of various grades of cutbacks as per ISI specifications are given in Table 6.7.

**Table 6.7 Viscosity of Cutbacks**

| Type and grade of Cutback | Viscosity in seconds in tar viscometer | | |
|---|---|---|---|
| | 4 mm orifice 25°C | 10 mm 25°C | 10 mm 40°C |
| RC-0, MC-O & SC-0 | 25 to 75 | | |
| RC-1, MC-1 & SC-1 | 50 to 100 | | |
| RC-2, MC-2 & SC-2 | | 10 to 20 | |
| RC-3, MC-3 & SC-3 | | 25 to 75 | |
| RC-4, MC-4 & SC-4 | | | 14 to 45 |
| RC-5, MC-5 & SC-5 | | | 60 to 100 |

Thus lower grade cutbacks like RC-0, RC-1 etc. would contain high proportion of solvent when compared with higher grades like RC-4 or RC-5. RC-0 and MC-0 may contain approximately 45 percent solvent and 55 percent bitumen, whereas, RC-5 and MC-5 may contain approximately15 percent solvent and 85 percent bitumen.

*Rapid Curing Cutbacks* are bitumens, fluxed or cutback with a petroleum distillate such as nephta or gasoline which will rapidly evaporate after using in construction, leaving the bitumen binder. The grade of the R.C. cutback is governed by the proportion of the solvent used. The penetration value of residue from distillation up to 360°C of RC cutback bitumen is 80 to 120.

*Medium curing cutbacks* are bitumen fluxed to greater fluidity by blending with a intermediate-boiling-point solvent like kerosene or light diesel oil. MC cutbacks evaporate relatively at slow rate because the kerosene-range solvents will not evaporate rapidly as the gasoline-range solvents used in the manufacture of RC cutbacks. Hence the designation 'medium curing' is given to this cutback type. MC products have good wetting properties and so satisfactory coating of fine grain aggregate and sandy soils is possible.

*Slow curing cutbacks* are obtained either by blending bitumen with high-boiling-point gas oil, or by controlling the rate of flow and temperature of the crude during the first cycle of refining. SC cutbacks or wood soils harden or set way slowly as it is a semi volatile material.

Various tests carried out on cut-backs bitumen are :

(a) Viscosity tests at specified temperature using specified size of orifice.

(b) Distillation test to find distillation fractions, up to specified temperature and to find the residue from distillation up to 360°C

(c) Penetration test, ductility test and test for matter soluble in carbon disulphide on residue from distillation up to 360°C

(d) Flash point test on cutback using Pensky Martens closed type apparatus.

### 6.3.6 Bituminous Emulsion

A bitumen emulsion is 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 fine globules in the other.

Usually, 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 formation of dispersion and to keep the globules of dispersed binder in suspension. The function of this emulsifier is to form a protective coating around the globules of binder resisting the coalescence of the globules. Emulsifiers usually adopted are soaps, surface active agents and colloidal powders. Half to one percent emulsifier by weight of finished emulsion are usually taken while preparing normal road emulsions. The bitumen/tar content of emulsions range from 40 to 60 percent and the remaining portion is water. The average diameter of globules of bitumen portion is about two microns.

Usually the bitumen grades which are emulsified for road construction works are those with penetration values between 190 and 320. Emulsions of tar and tar bitumen mixture are also prepared, but their use is restricted. Two methods commonly followed for the preparation of emulsions are the colloidal mill method and the high-speed mixer method. The manufactured emulsions are stored in air tight drums.

When the emulsion is applied on the road, it breaks down and the binder starts binding 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 to black. If the bitumen emulsion is intended to break rapidly, the emulsion is said to possess rapid-set quality. Emulsions which do not break spontaneously on contact with stone, but break during mixing or by fine mineral dust are medium-set grades. When special types of emulsifying agents are used to make the emulsion relatively stable, they are called slow setting grades.

Emulsions are used in bituminous road constructions, especially in maintenance and patch repair works. The main advantage of emulsion is that it can be used in wet weather even when it is raining. Also emulsions have been used in soil stabilization, particularly for the stabilization of sands in desert areas.



Some of the general properties of road emulsions are judged by the following tests :

(i) *Residue on Sieving* : It is desirable to see that not more than 0.25 percent by weight of emulsion consists of particles greater than 0.15 mm diameter.

(ii) *Stability to Mixing with Coarse Graded Aggregate* : This test carried out to find if the emulsion breaks down and coats the aggregate with bitumen too early before mixing is complete.

(iii) *Stability to Mixing with Cement* : This test is carried out to assess the stability of emulsions when the aggregate contains large proportions of fines.

(iv) *Water Cement* : To know the percentage water in the emulsion which depends on the type of the emulsion.

(v) *Sedimentation* : Some sedimentation may occur when a drum of emulsion is left standing before use, but on agitation, the emulsion redisperses and can be used.

(vi) *Viscosity* : The viscosity of emulsified bitumen should be low enough to be sprayed through jets or to coat the aggregates in simple mixing.

Three types of bituminous emulsion are prepared, viz., (i) Rapid Setting (RS), (ii) Medium Setting (MS) and (iii) Slow Setting (SS) types. Rapid Setting type emulsion is suitable for surface dressing and penetration macadam type of construction. Medium Setting type is used for premixing with coarse aggregates and Slow Setting type emulsion is suitable for fine aggregate mixes.

### 6.3.7 Tar

Tar is the viscous liquid obtained when natural organic materials such as wood and coal carbonized or destrictively 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 is more widely used for road work because it is superior. 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 the desired road tar.

There are five grades of roads tars, viz., RT-1, RT-2, RT-3, RT-4 and RT-5, based on their viscosity and other properties. RT-1 has the lowest viscosity and 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 premixing chips for top course and light carpets. RT-4 is generally used for premixing tar macadam in base course. For grouting purposes RT-5 may be adopted, which has the highest viscosity among the road tars.

The various tests that are carried out on road tars are listed below :

(i) Specific gravity test

(ii) Viscosity test on standard tar viscometer

(iii) Equiviscous temperature (EVT)

(iv) Softening point

(v) Softening point of residue

(vi) Float test

(vii) Water content

(viii) Distillation fraction on distillation upto 200°C, 200°C to 270°C and 270°C to 330°C.

(ix) Phenols, percent by volume

(x) Naphthalene, percent by weight

(xi) 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 the ISI. The summary of the important test properties is given in Table 6.8.

**Table 6.8 Properties and Requirements of Road Tars**

| Sl. No. | Property | Road Tar Grades | | | | |
|---|---|---|---|---|---|---|
| | | RT-1 | RT-2 | RT-3 | RT-4 | RT-6 |
| 1. | Viscosity by standard tar viscometer (10 mm) | | | | | |
| | (a) at temperature, °C | 35 | 40 | 45 | 55 | - |
| | (b) viscosity range, seconds | 33-55 | 30-55 | 35-60 | 40-60 | - |
| 2. | Equiviscous Temperature (EVT) range, °C | 32-36 | 37-41 | 43-46 | 53-57 | 63-67 |
| 3. | Softening point, °C | - | - | - | - | 45-50 |
| 4. | Specific gravity range at 27°C | 1.16-1.26 | 1.16-1.26 | 1.18-1.28 | 1.18-1.28 | 1.18-1.28 |

*Comparison of Tar and Bitumen*

Bitumen and tar have black to dark brown colour. But bitumen is a petroleum product whereas tar is produced by the destructive distillation of coal or wood. The chemical constituents of bitumen and tar are quite different. Bitumen is soluble in carbon disulphide and in carbon tetrachloride; but tar is soluble only in toluene. The coats aggregate more easily and retains it better in presence of water than bitumen. But the tar is considered to have much inferior weather resisting property. Tar is more temperature susceptible, resulting in a great variation in viscosity with temperature . Bitumens are less temperature susceptible. The free carbon content is more in tar as seen from the solubility test.

## 6.4 BITUMINOUS PAVING MIXES

### 6.4.1 Requirements of Bituminous Mixes

The mix design should aim at an economical blend, with proper gradation of aggregates and adequate proportion of bitumen so as to fulfil the desired properties of the mix. Bituminous concrete or asphaltic concrete is one of the highest and costliest type of flexible pavement layers used in the surfacing course. The desirable properties of a good bituminous mix are stability, durability, flexibility, skid resistance and workability.

*Stability* is defined as resistance of the paving mix to deformation under load and that it is a stress which causes a specified strain depending upon anticipated field conditions. Stability is a function of friction and cohesion. Frictional resistance is a function of both inter-particle 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 correlated terms. If voids are restricted, the resulting strength property of the paving mixes improve. Minimum voids requirement qualified for a given mix should be so selected which would provide space for necessary densification that may develop under traffic movements 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 and abrasive actions. Weathering causes hardening and this depends upon loss of volatiles and oxidation. Tensile strain is introduced in the top layer consisting of bituminous mix when wheel loads ply over it. Excessive strain causes cracking or plastic failure.

*Flexibility* is a property of the mix that measures the level bending strength. Thus suitability of the given bituminous paving mix need the consideration of all the factors listed above. *Skid resistance* is defined as the resistance of the finished pavement against skidding and is a function of surface texture and bitumen content. Workability is the ease with which the mix can be laid and compacted. It is function of gradation of aggregates, their shape and texture, bitumen content and its type.

Mix design methods should aim at determining the properties of aggregates and bituminous material which would give a mix having the following properties.

(i) Sufficient stability to satisfy the service requirements of the pavement and the traffic conditions, without undue displacements.

(ii) Sufficient bitumen to ensure a durable pavement by coating the aggregate and bonding them together and also by water-proofing the mix.

(iii) Sufficient voids in the compacted mix as to provide a reservoir space for a slight amount of additional compaction due to traffic and to avoid flushing, bleeding and loss of stability.

(iv) Sufficient flexibility even in the coldest season to prevent cracking due to repeated application of traffic loads.

(v) Sufficient workability while placing and compacting the Mix.

(vi) The mix should be the most economical one that would produce a stable, durable and skid resistant pavement.

Three mix design methods, namely, Marshall, Hveem and Hubbard-Field methods are briefly explained in the following paragraphs. These methods have been widely used by various agencies in the design and construction with satisfactory results. In each of these methods, the laboratory test results on the mixes have been correlated with the performance studies in developing the design criteria.

### 6.4.2 Design of Bituminous Mixes

The following steps may be followed for a rational design of a bituminous mix.

*(i) Selection of Aggregate*

Aggregates which possess sufficient strength, hardness, toughness and soundness are chosen, keeping in view the availability and economic consideration. Crushed aggregates and sharp sands produce higher stability of the mix when compared with gravel and rounded sands.

*(ii) Selection of Aggregate Grading*

The properties of a bituminous mix including the density and stability are very much dependent on the aggregates and their grain size distribution.

Most of the agencies and engineering organisations have specified the use of dense graded mixes and they do not prefer the open grading. As higher maximum size of aggregate gives higher stability, usually the biggest size that can be adopted keeping in view of the compacted thickness of the layer is selected, provided all other factors are equal. In base course maximum aggregate size of 2.5 to 5 cm are used whereas for surface course 1.25 to 1.87 cm size are used in the mixes.

The gradation of final mix after blending of the aggregates and filler should be within the specified range. The gradation for 40 mm thick bituminous concrete surface course specified by the IRC is given in Table 6.9.

**Table 6.9 Specified Gradation of Aggregates for Bituminous (Asphaltic) Concrete Surface Course**

| Sieve size, mm | Percent passing, by weight | |
|---|---|---|
| | Grade 1 | Grade 2 |
| 20 | - | 100 |
| 12.5 | 100 | 80 – 100 |
| 10.0 | 80 – 100 | 70 – 90 |
| 4.75 | 55 – 75 | 50 – 70 |
| 2.36 | 35 – 50 | 35 – 50 |
| 0.600 | 18 – 29 | 18 – 29 |
| 0.300 | 13 – 23 | 13 – 23 |
| 0.150 | 8 – 16 | 8 – 16 |
| 0.075 | 4 – 10 | 4 – 10 |
| Binder content, percent by weight of mix | 5 – 7.5 | 5 – 7.5 |

*(iii) Determination of Specific gravity*

The specific gravity of the bituminous material is not usually determined, if already known. The specific gravity of aggregates are represented as either bulk specific gravity, or apparent specific gravity, or effective specific gravity. In bulk specific gravity the overall volume of the aggregate is taken. In apparent specific gravity the volume of capillaries which are filled by water on 21 hours soaking is excluded. When either bulk or apparent specific gravity is used, the specific gravity of such aggregate (i.e., coarse, fine and filler) is found and that of the combined aggregate is calculated. But when effective specific gravity is used, the specific gravity of the total or combined aggregate is determined and the average specific gravity $G_a$ of blended aggregate mix is calculated from the equation,

$$G_a = \frac{100}{W_1/G_1 + W_2/G_2 + W_3/G_3 + W_4/G_4} \qquad (6.9)$$

Where $W_1$, $W_2$, $W_3$, $W_4$ are percent by weight of aggregate 1, 2, 3 and 4; $G_1$, $G_2$, $G_3$ and $G_4$ are the specific gravities of the respective aggregates.

*(iv) Proportioning of Aggregates*

First the design grading is decided based on the type of the construction work, thickness of the layer and the availability of aggregates. Then the available aggregates are proportioned either by analytical method, or graphical method, or by trial and error basis from laboratory test. Two of the graphical methods of proportioning, viz. triangular chart method and Rothfutch's method, are explained in Chapter 9.



*(v) Preparation of Specimen*

The preparation of specimen depends on the stability test method employed. Hence the size of the specimen, compaction and other specification should be followed as specified in the stability test method. The stability test methods which are in common use for the design mix are, Marshall, Hubbard-Field and Hveem. Hence after deciding the test methods, the specimens are moulded as per specification.

*(vi) Determination of Specific Gravity of Compacted Specimen*

The specific gravity of the compacted specimens, as moulded is determined. With known values of specific gravity of aggregate and bitumen, the theoretical maximum specific gravity of the mix $G_t$ is calculated for a given bitumen content, using the formula,

$$G_t = \frac{100}{(100 - W_b)/G_a + W_b/G_b} \qquad (6.10)$$

where $W_b$ = percent by weight of bitumen content

$G_b$ = specific gravity of bitumen

$G_a$ = average specific gravity of aggregates.

The *theoretical density* $\gamma_t$, percent solids by volume is calculated from the formula.

$$\gamma_t = \frac{100\,G}{G_t}$$

Here, $G$ = actual specific gravity of test specimen

$G_t$ = theoretical maximum specific gravity

Hence *percent air voids* in the specimen,

$$V_v = 100 - \gamma_t = \frac{100\,(G_t - G)}{G_t}$$

The *Voids in the Mineral Aggregate* (VMA) is calculated from the equation :

$$\text{Volume VMA} = (V_v + V_b) = 100 - \frac{G}{W_a} \qquad (6.11)$$

Here $V_b$ is the percentage of bitumen and $W_a$ is aggregate content percent by weight

Hence, *Percent Voids Filled with Bitumen* (VFB)

$$= \frac{100\,V_b}{VMA} \qquad (6.12)$$

*(vii) Stability Tests on Compacted Specimens*

One of the stability tests is carried out based on the design method selected.

*(viii) Selection of Optimum Bitumen Content*

The optimum bitumen content is selected based on the test method adopted and the design requirements considered.

The Marshall, Hubbard-Field and Hveem stability test methods are explained here.

**Marshall method of bituminous mix design**

*Bruce Marshall*, formerly Bituminous Engineer with Mississippi State Highway Department formulated Marshall method for designing bituminous mixes. Marshall's test procedure was later modified and improved upon by U. S. Corps of Engineer through their extensive research and correlation studies. ASTM and other agencies have standardized the test procedure. Generally, this stability test is applicable to hot-mix design of bitumen and aggregates with maximum size 2.5 cm. In India, bituminous concrete mix is commonly designed by Marshall method.

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 a rate of 5 cm per minute. The test procedure is used in the design and evaluation of bituminous paving mixes. The test 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 test

The stability of the mix is defined as a maximum load carried by a compacted specimen at a standard test temperature of 60°C. The flow is measured as the deformation in units of 0.25 mm between no load and maximum load carried by the specimen during stability test. (The flow value may also be measured by deformation units of 0.1 mm). In this test an attempt is made to obtain optimum binder content for the aggregate mix type and traffic intensity.

The apparatus consists of a cylindrical mould, 10.16 cm diameter and 6.35 cm height, with a base plate and collar. A compaction pedestal and hammer are used to compact a specimen by 4.54 kg weight with 45.7 cm height of fall. A sample extractor is used to extrude the compacted specimen from the mould. A breaking head is used to test the specimen by applying a load on its periphery perpendicular to its axis in a loading machine of 5 tonnes capacity at a rate of 5 cm per minute. A dial gauge fixed to the guide rods of the testing machine serves as flow meter to measure the deformation of the specimen during loading.

The coarse aggregates, fine aggregates and filler material should be proportioned and mixed in such a way that the final mix after blending has gradation within the specified range as given in Table 6.9. Approximately 1200 g of the mixed aggregates and the filler are taken and heated to a temperature of 175 to 190°C. The bitumen is heated to a temperature of 121 to 145°C and the required quantity of the first trial percentage of bitumen (say, 3.5 or 4.0 percent by weight of material aggregates) is added to the heated aggregates and thoroughly mixed at the desired temperature of 154 to 160°C. The mix is placed in a pre-heated mould and compacted by a rammer with 50 blows on either side at temperature of 138 to 149°C. (Suitable heating, mixing and compacting temperatures are chosen depending upon the grade of the bitumen). The weight of the mixed aggregate taken for the preparation of the specimen may be suitably altered to obtain a compacted



thickness of 63.5 ± 3.0 mm. Three or four specimens may be prepared using each trial bitumen content. The compacted specimens are cooled to room temperature in the moulds and then removed from the moulds using a specimen extractor. The diameter and mean height of the specimens are measured and then they are weighed in air and also suspended in water. The specimens are kept immersed in water in a thermostatically controlled water bath at 60 ± 1°C for 30 to 40 minutes. The specimens are taken out one by one, placed in the Marshall test head and tested to determine Marshall Stability Value which is the maximum load in kg before failure and the Flow Value which is the deformation of the specimen in 0.25 mm units upto the maximum load. The corrected Marshall Stability Value of each specimen is determine by applying the appropriate correction factor, if the average height of the specimen is not exactly 63.5 mm; the correction factors to be applied are given in Table 6.10.

**Table 6.10 Correction Factors for Marshall Stability Values**

| Volume of specimen in cc | Thickness of specimen in mm | Correction factor |
|---|---|---|
| 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 |

The above procedure is repeated on specimens prepared with other values of bitumen content, in suitable increments, say 0.5 percent, out about 7.5 or 8.0 percent bitumen by weight of total mix. The bulk density, percent air voids, voids in mineral aggregates and voids filled with bitumen are calculated using the following relationships.

*Percent Air Voids*

$$V_v = \frac{G_t - G_m}{G_m} \times 100 \tag{6.13}$$

Here $G_m$ = bulk density or mass density of the specimen

$G_t$ = theoretical specific gravity of mixture

$$G_t = \frac{1000}{\frac{W_1}{G_1} + \frac{W_2}{G_2} + \frac{W_3}{G_3} + \frac{W_4}{G_4}}$$

where $W_1$ = percent by weight of coarse aggregate in total mix

$W_2$ = percent by weight of fine aggregate in total mix

$W_3$ = percent by weight of filler in total mix

$W_4$ = percent by weight of bitumen in total mix

$G_1$ = Apparent specific gravity of coarse aggregate

$G_2$ = Apparent specific gravity of fine aggregate

$G_3$ = Apparent specific gravity of filler

$G_4$ = Specific gravity of bitumen

*Percent Voids in Mineral Aggregate (VMA)*

$$VMA = V_v + V_b$$

Here $V_v$ = volume of air voids, %

$$V_b = \text{volume of bitumen, \%} = G_m = \frac{W_4}{G_4}$$

*Percent Voids Filled with Bitumen (VFB)*

$$VFB = \frac{100\, V_b}{VMA} \qquad (6.14)$$

The average value of each of the above properties are found for each mix with the different bitumen contents. Graphs are plotted with the bitumen content on the X-axis and the following values on the Y-axis.

(i) Marshall stability value

(ii) Flow value

(iii) Unit weight

(iv) Percent voids in total mix ($V_v$)

(v) Percent voids filled with bitumen (VFB)

Typical plots of these are shown in Fig. 6.27.

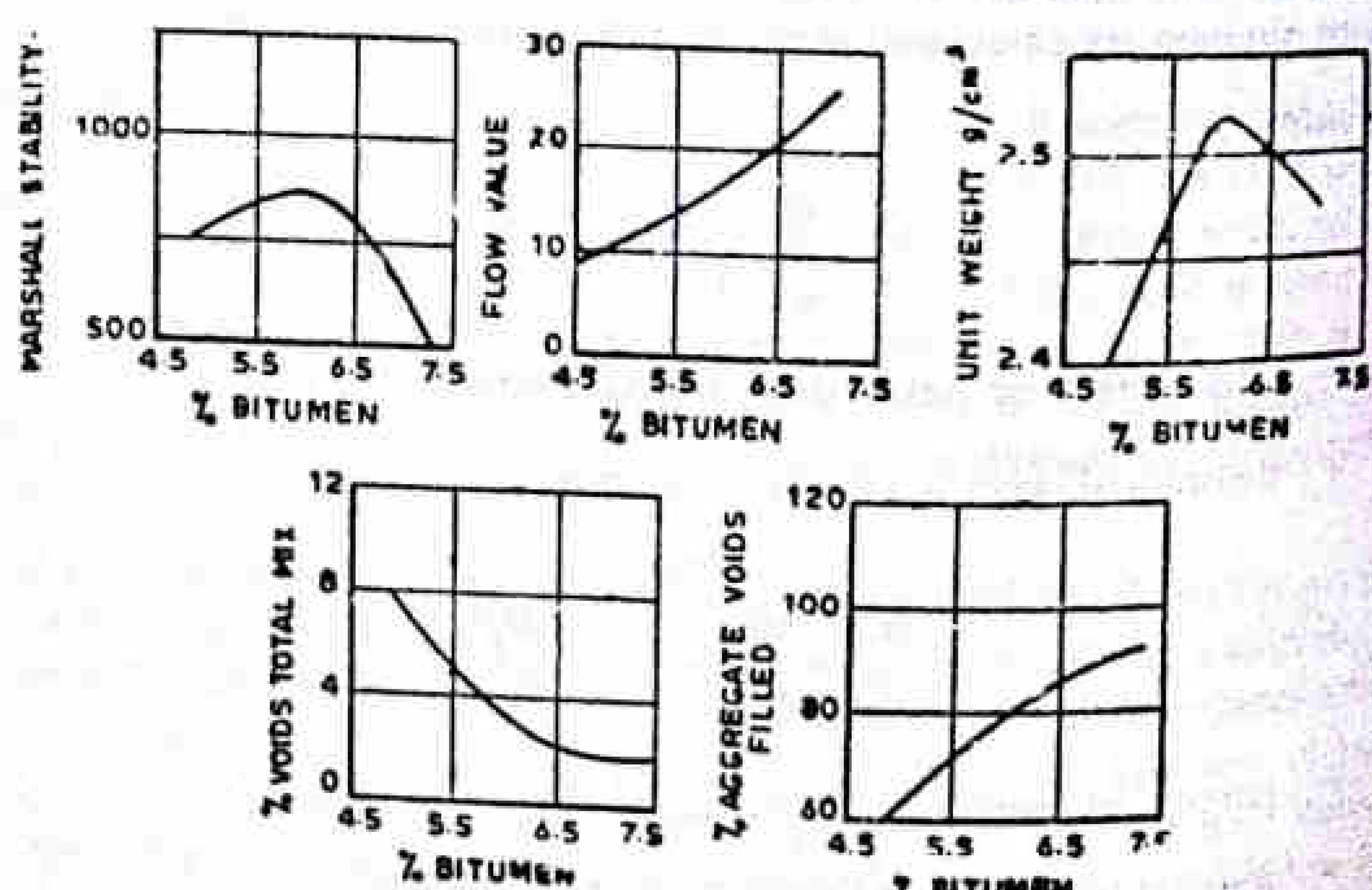



**Fig. 6.27 Bituminous Mix Design by Marshall Test**

The optimum bitumen content for the mix design is found by taking the average value of the following three bitumen contents found from the graphs of the test results.

(i) Bitumen content corresponding to maximum stability.



(ii) Bitumen content corresponding to maximum unit weight.

(iii) Bitumen content corresponding to the median of designed limits of percent air voids in total mix (4%).

The Marshall Stability value, Flow value and percent Voids Filled with Bitumen at the average value of bitumen content are checked with the Marshall mix design criteria/specifications, given in Table 6.11.

**Table 6.11 Marshall Mix Design Criteria for Bituminous concrete**

| Test property | Specified value |
|---|---|
| Marshall Stability, kg | 340 (minimum) |
| Flow value, 0.25 mm units | 8 to 16 |
| Air voids in total mix, $V_v$% | 3 to 5 |
| Voids filled with bitumen, VFB% | 75 to 85 |

Mixes with very high Marshall stability values and low Flow values are not desirable as the pavements constructed with such mixes are likely to develop cracks due to heavy moving loads, if the pavements components permit relatively high deflection values.

**Example 6.3**

Find the optimum bitumen content of a mix, considering the data plotted in Fig. 6.27.

**Solution**

Here the optimum bitumen contents corresponding to maximum values of stability and unit weight are 5.5 and 6 percent respectively, from Fig. 6.27.

Bitumen content corresponding to 4% air voids in total mix is 5.8 percent.

Hence the optimum bitumen content for the mix design is the average of the three values

$$= \frac{5.5 + 6 + 5.8}{3} = 5.8 \text{ percent}$$

The Flow value corresponding to 5.8 percent bitumen is 15, which is satisfactory as per Table 6.11. VFB at 5.8% bitumen is 78%, which is also as per the design criteria.

**Modified Hubbard-Field method of bituminous mix design**

The method was developed by *P. Hubbard and F. C. Field.* The original method was intended to design sheet asphalt mix. Later the method was modified for the design of bituminous mixes having coarse aggregate upto 19 mm size.

The equipment consists of 15.24 cm diameter mould and other compacting equipment including tampers and compression machine of capacity 5000 kg. There is a testing assembly consisting of a ring of internal diameter 14.6 cm through which the specimen is extruded by applying load through the compression machine. The assembly is shown in Fig. 6.28.

For the desired blend and gradation of the aggregates, batch weights are calculated for producing specimens of compacted size, 15.2 cm diameter and 7 to 7.6 cm height. The weighed aggregates, filter and the bituminous materials are heated to the prescribed

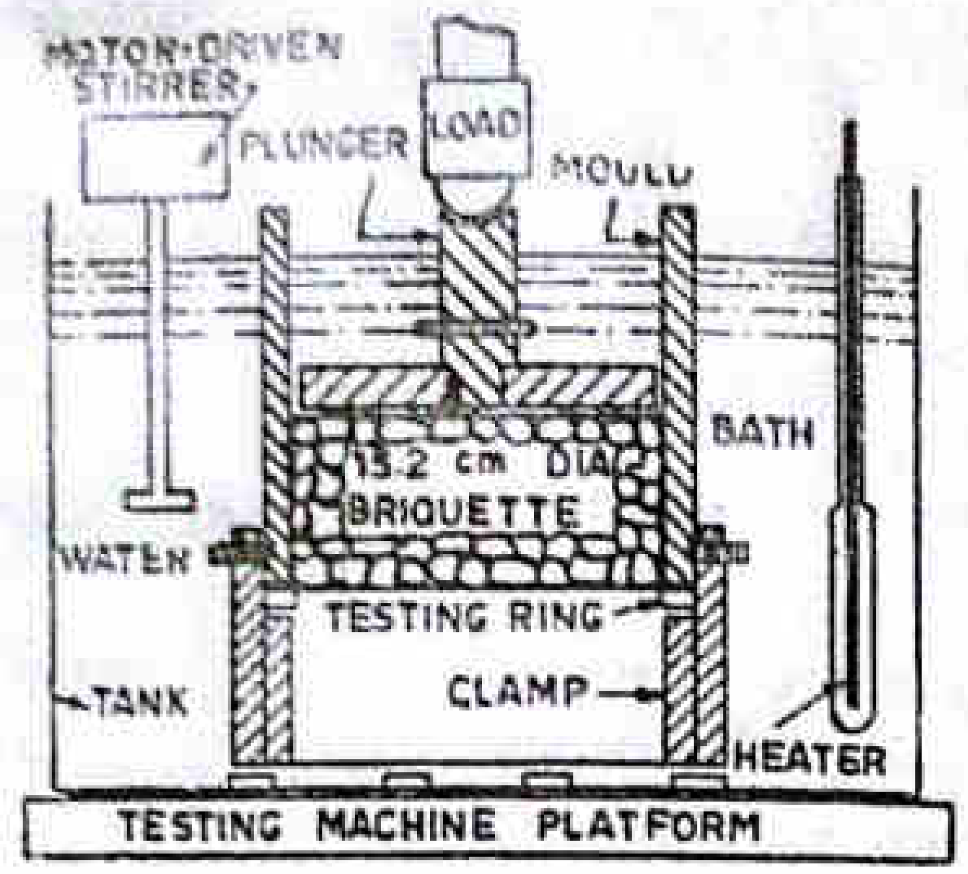



**Fig. 6.28 Modified Hubbard-Field Test Set-up**

temperature, mix placed in the preheated mould and tamped in two layers by 30 blows each with the specified tampers. The specimen is tamped again on the reverse side by 30 blows by each of the two tampers. Then a static load of 4536 kg is applied on the specimen for two minutes and the specimen is cooled in water to temperature less than 37.8°C, maintaining the same compressive load. The specimen is then removed, weighed and measured.

The specimen is placed in the test mould assembly over the test ring of internal diameter of 14.6 cm, and the plunger is loaded on the top of the specimen. The entire assembly is kept in a water bath maintained at 60°C for atleast one hour in position under the compression machine. The compressive load is applied at a constant rate of deformation of 6.1 cm per minute and the maximum load in kg developed during the test is recorded as the stability value. The average stability value of all the specimens tested using a particular mix is found. The tests are repeated with other bitumen contents as in Marshall method.

For each bitumen content the average value of specific gravity, percent voids in total mix and percent aggregate voids are calculated. The following graphs are plotted :

(i) Stability versus bitumen content

(ii) Unit weight versus bitumen content

(iii) Percent voids in total mix versus bitumen content.

(iv) Percent aggregate voids versus bitumen content.

Typical plots of these are shown in Fig. 6.29. The following criteria have been specified by the Asphalt Institute for the design of bituminous mix.

| The property | Medium and light traffic | Heavy and very heavy traffic |
|---|---|---|
| Stability, kg | 545 – 910 | > 910 |
| Voids, total mix, % | 2 – 5 | 2 – 6 |

For determining the optimum bitumen content, first the bitumen content corresponding to 3 or 3.5 percent voids in total mix is found from the graph. The corresponding stability is read from the stability curve. If the stability values are within



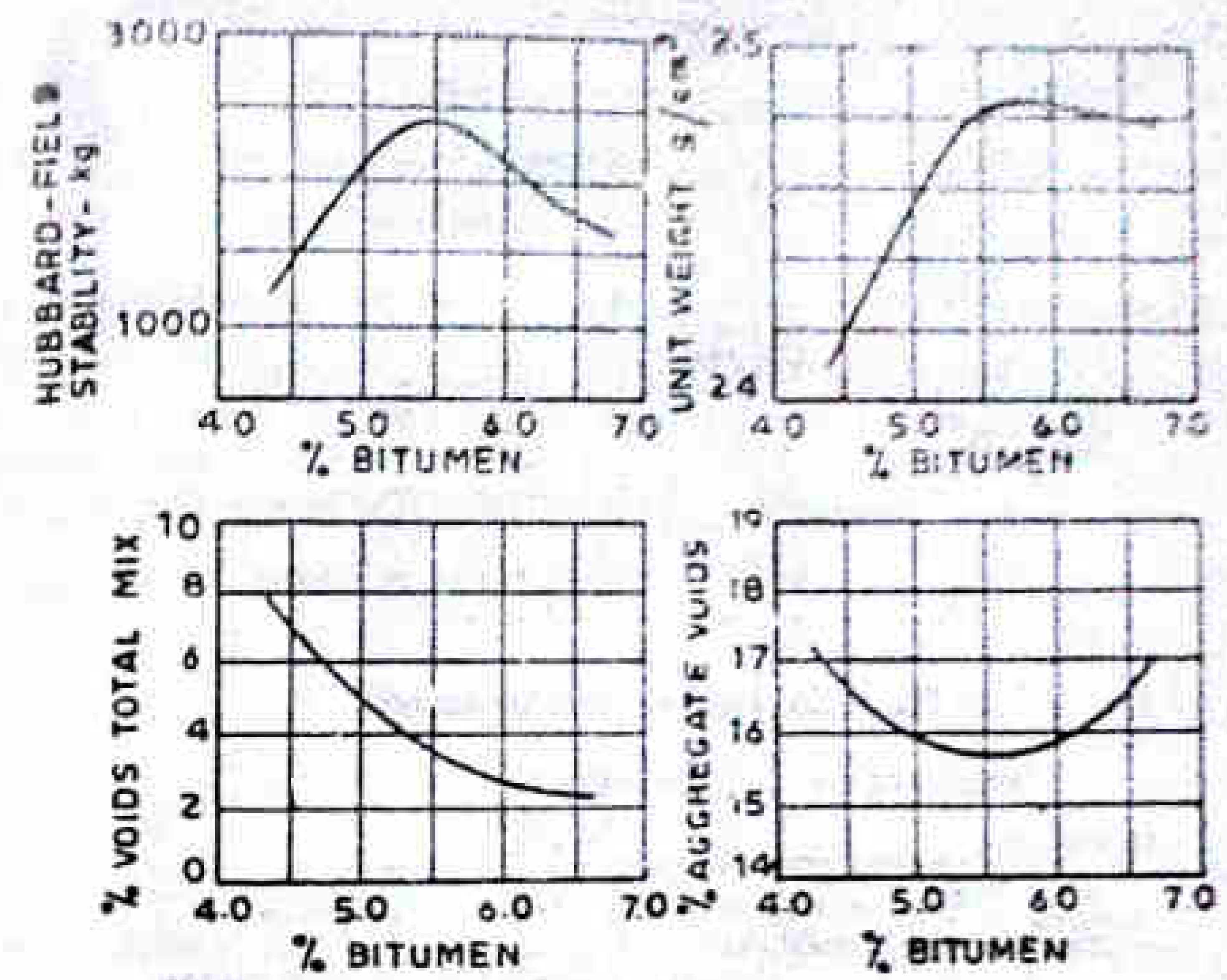



**Fig. 6.29 Mix Design by Modified Hubbard Method**

the specified limits, mix is satisfactory. If both stability and void requirements are not satisfied by a mix, the mix should be redesigned to correct the deficiency. The final selection of the mix design should be based on economics and suitability of the mix from the test requirements.

**Example 6.4**

Find the optimum bitumen content of the mix considering data plotted in Fig. 6.29.

**Solution**

Bitumen content corresponding to 3.5 percent voids in total mix is 5.5 percent. The Hubbard Field stability at 5.5 percent bitumen is 2300 kg and hence the mix is satisfactory for very heavy traffic.

**Hveem method of bituminous mix design**

This method was developed by *Francis N. Hveem*, Materials and Research Engineer for the California Division of Highways.

The equipment consists of compaction mould 10 cm inside diameter and 12.7 cm height. The mechanical compactor is a kneading compactor capable of exerting a force of 35 kg/cm$^2$ under the tamper foot. The swell test assembly consist of dial gauge and tripod. The Hveem stabilometer loading equipment and cohesiometer are shown in Fig. 6.30 and 6.31.

*Stabilometer Test*

The stabilometer consists of a cylindrical mould which can accommodate a specimen 10 cm diameter and 6.25 cm height resting over a rigid metal cylinder. The specimen is encased in the rubber membrane which acts as an inner wall of the mould. Fluid pressure

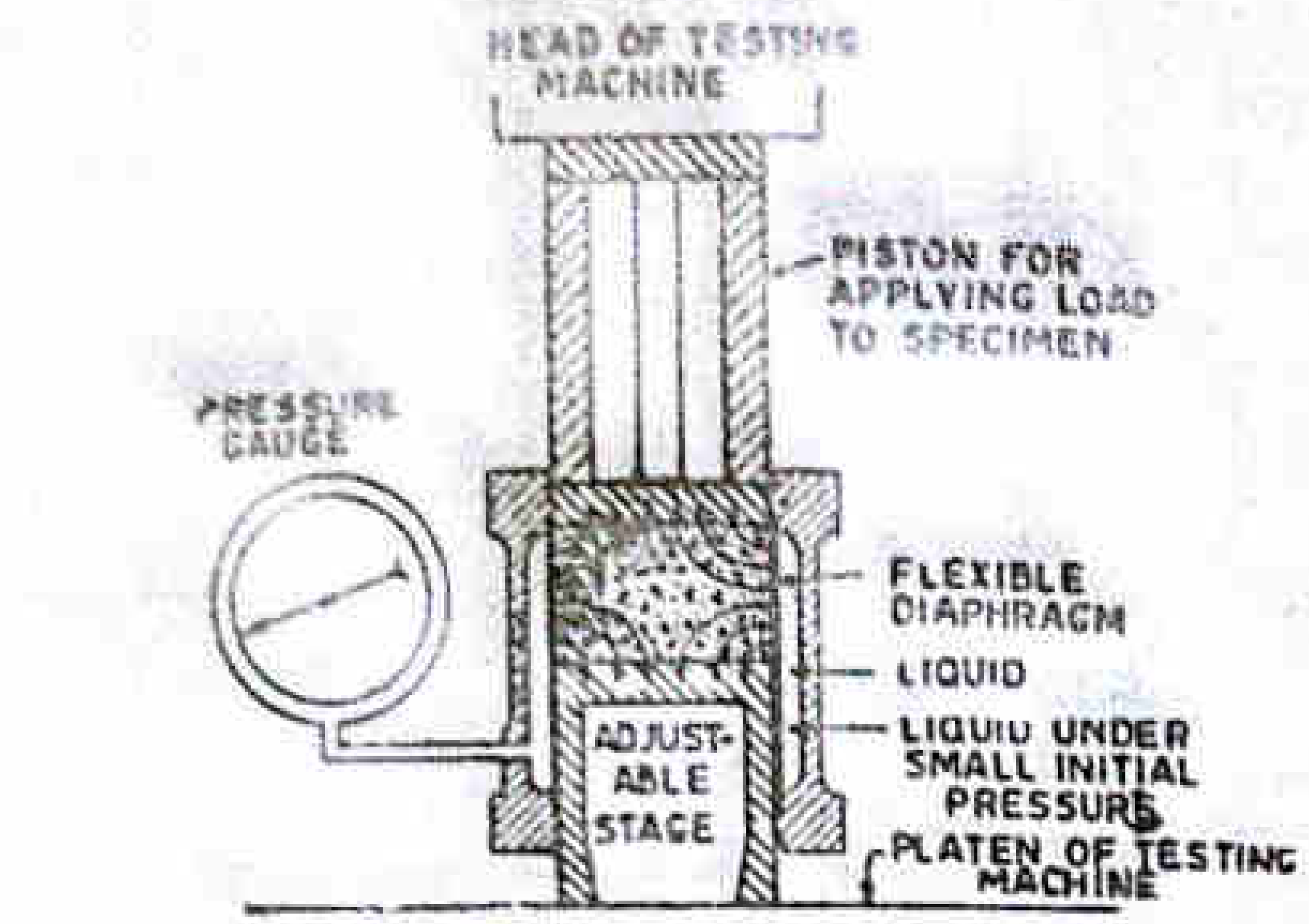



Fig. 6.30 Haveem Stabilometer

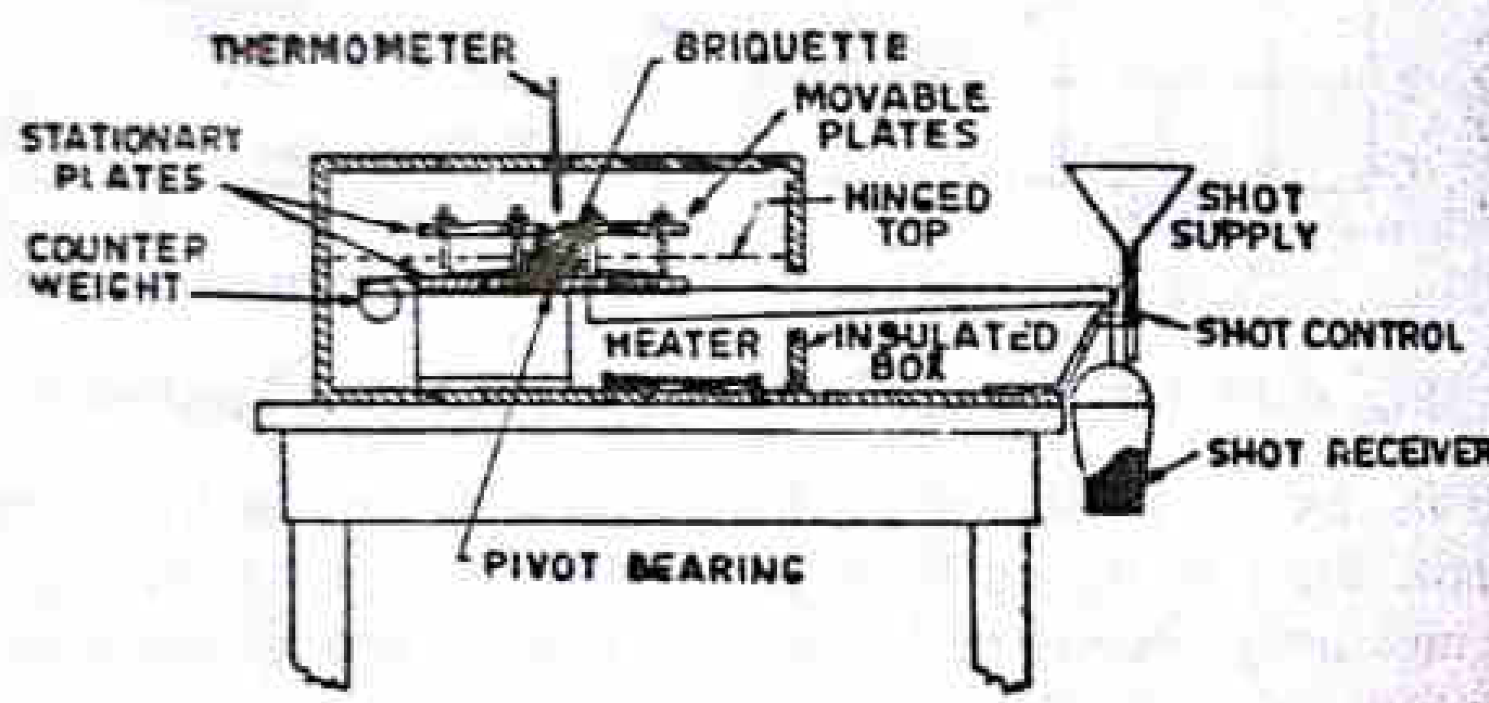



Fig. 6.31 Cohesiometer

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. The vertical pressure is applied through the loading head placed on the loading machine.

As per the requirement of the project, the aggregate gradation blend are chosen. The optimum bitumen content is then estimated using the *Centrifuge Kerosene Equivalent* (C.K.E.) method. (The percentage of kerosene retained in the aggregate after being soaked and centrifuged as specified is called C.K.E. value). Charts are available to find the optimum bitumen content from the C.K.E. value.

For mix design, specimens are prepared with three bitumen contents, one equal to 0.5 to 1.0 percent above and one 0.5 to 1.0 percent below the estimated optimum bitumen content. Two additional specimens are prepared using the estimated optimum bitumen content, for the swell tests. The specimens are compacted at 110°C using a kneading compactor, with a circular ram the pressure of which increases without impact upto 35 $kg/cm^2$, maintained for about 0.4 seconds and then released.

The compacted swell test specimens are cooled at room temperature for one hour and placed in the water pan for 24 hours after setting up the swell measuring dial arrangement. The swell is measured to the nearest 0.025 mm, and is represented as such.

The stabilometer test specimens are kept at temperature of 60°C and held in position in the Hveem Stabilometer. The fluid pressure is raised to 0.35 $kg/cm^2$ by the



displacement pump handle and the valve is closed. Vertical loads are then applied in sequence of 227, 454 and in increments of 454 kg up to a maximum of 2722 kg. The displacement valve is opened and the pressure is adjusted to 0.35 $kg/cm^2$.

The handle of the pump is rotated at two turns per second till the lateral pressure increases to 7 $kg/cm^2$ and the number of turns is noted.

The specimen from the stabilometer is recovered after releasing the pressures, weighed and measured to determine the bulk density. The sample is then maintained at a temperature of 60°C for two hours and placed in the cohesiometer, the cabinet of which is also maintained at a temperature of 60°C. The led shots are allowed to flow at a rate of 1800 g per minute until the specimen breaks, and the led shots in the bucket is weighed (L grams).

The stabilometer and cohesiometer value are calculated using the following formulae :

$$S = \frac{22.2}{\frac{P_h - D_2}{P_v - P_h} + 0.222} \tag{6.15}$$

Here,

$S$ = relative stability

$P_v$ = vertical pressure at 28 $kg/cm^2$ or at a total load of 2268 kg.

$P_h$ = horizontal pressure corresponding to $P_v = 28\ kg/cm^2$.

$D_2$ = Displacement on specimen represented as number of turns of pump handle to raise $P_h$ from 0.35 to 7 $kg/cm^2$.

$$C = \frac{L}{W(0.2H + 0.0176H^2)} \tag{6.16}$$

Here

$C$ = Cohesiometer value

$L$ = weight of shots in gm

$W$ = diameter or width of specimen in cm

$H$ = height of specimen in cm

Using the specific gravity of the test specimens and the apparent specific gravity of aggregates the percent voids in the total mix is calculated.

The suitability of the hot mix is decided based on the design criteria given in Table 6.12.

**Table 6.12 Design Criteria by Hveem Method**

| Test value | Criteria | | |
|---|---|---|---|
| | Light traffic | Medium traffic | Heavy traffic |
| Stabilometer value, R | > 30 | > 35 | > 37 |
| Cohesiometer value, C | > 50 | > 50 | > 50 |
| Swell, mm | < 0.76 | < 0.76 | < 0.76 |
| Air void, percent | > 4 | > 4 | > 4 |

The stabilometer is also employed in evaluating the resistance value (R-value) of soil subgrade material. The compaction is done using a kneading compactor with 24.6 kg per cm$^2$ pressure, 100 times. After the compaction, a load is applied at a rate of 907 kg per minute to record the exudation pressure required to force water out of the specimen. Expansion pressure is also noted permitting the specimen to remain in water for 16 to 20 hours.

The stabilometer resistance R-values is determined by placing the specimen in the stabilometer and applying the lateral and vertical pressures as specified. The R-value of soil is calculated from the formula :

$$R = 100 - \frac{100}{\frac{2.5}{D_2}\left(\frac{P_v}{P_h} - 1\right) + 1} \quad (6.17)$$

Here,

$P_v$ = vertical pressure applied (11.2 kg/cm$^2$).

$P_h$ = horizontal pressure transmitted at $P_v$ = 11.2 kg/cm$^2$.

$D_2$ = displacement of stabilometer fluid necessary to increase the horizontal pressure from 0.35 to 7 kg/cm$^2$, measured in number of revolutions of the calibrated pump handle.

R-value of soil is used in the California Resistance value method of pavement design given in Art. 7.3.4.

## 6.5 PORTLAND CEMENT AND CEMENT CONCRETE

Portland cement is used in the construction of cement concrete pavements. Cement concrete pavements are considered to be the highest pavement type which withstand heavy traffic even under adverse subgrade and climatic conditions.

Portland cement is also being used in soil-cement stabilization for the construction of stabilized sub-base and base courses. The properties of cement and cement concrete and the mix design details of cement concrete are beyond the scope of this book and hence have not been included here.

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