MODULE – 3
PAVEMENT MATERIALS
INTRODUCTION

Subgrade Soil

Subgrade soil is an integral part of the road pavement structure which directly receives
the traffic load from the pavement layers. 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.

Desirable Properties

The desirable properties of soil as a highway material are

a) Stability
b) Incompressibility
c) Permanency of strength
d) Minimum changes in volume and stability under adverse conditions of weather and
ground water
e) Good drainage
f) 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 strength values of
the subgrade. 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.
SOIL CLASSIFICATION

Soil Classification Based on Grain Size

There are several soil classification systems based on grain size of soil, according to which soils have been classified as

- a) Gravel
- b) Sand
- c) Silt and Clay

The most widely accepted grain size classification system is MIT soil classification system. The Bureau of Indian Standards (BIS) has also adopted the same limits as MIT system for the Indian Standard Classification System for soil grains. The limits of grain size are as follows.

<table>
<thead>
<tr>
<th>Gravel</th>
<th>Sand</th>
<th>Silt</th>
<th>Clay</th>
</tr>
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<tbody>
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<td></td>
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</tbody>
</table>

**Very coarse soils**

- Boulder size: > 300 mm
- Cobble size: 80 - 300 mm

**Coarse soils**

- Gravel size (G): Coarse 20 - 80 mm; Fine 4.75 - 20 mm
- Sand size (S): Coarse 2 - 4.75 mm; Medium 0.425 - 2 mm; Fine 0.075 - 0.425 mm

**Fine soils**

- Silt size (M): 0.002 - 0.075 mm
- Clay size (C): < 0.002 mm

Highway Research Board (HRB) classification of soils

The Highway Research Board (HRB) soil classification method is also called Revised Public Roads Administration (PRA) soil classification system. With just three simple laboratory tests namely sieve analysis, liquid limit and plastic limit, it is possible to classify the soils. The HRB soil classification system is generally adopted in highway engineering for the classification of subgrade soils.

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 0.075 mm sieve being less than 35. A-4, A-5, A-6 and A-7, soils are fine grained or silt-clay soils, passing 0.075 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 up to 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 or 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 P1 greater than 10%). These soils have low permeability and high volume change properties with changes in moisture content.
CALIFORNIA BEARING RATIO (CBR) TEST

This is a penetration test developed by the California division of highway. For evaluating the stability of soil subgrade and other pavement materials. The test results have been correlated with flexible pavement thickness requirement for highway and airfield. CBR test may be conducted in the laboratory on a prepared specimen in a mould or in situ in the field.

Laboratory CBR test

The laboratory CBR apparatus consists of

Cylindrical mould

Mould 150mm dia, 175mm height with 50mm collar height, detachable perforated base with spacer disc of 148mm dia and 47.7mm thick is used to obtain a specimen of exactly 127.3mm height.

Loading Machine

Compression machine operated at a constant rate of 1.25mm/min. Loading frame with cylindrical plunger 50mm dia & dial gauge for measuring the deformation due to application of load.

Compaction rammer

<table>
<thead>
<tr>
<th>Type of compaction</th>
<th>No of layers</th>
<th>Wt of hammer (kg)</th>
<th>Fall (cm)</th>
<th>No of blows</th>
</tr>
</thead>
<tbody>
<tr>
<td>Light compaction</td>
<td>3</td>
<td>2.6</td>
<td>31</td>
<td>56</td>
</tr>
<tr>
<td>Heavy compaction</td>
<td>5</td>
<td>4.89</td>
<td>45</td>
<td>56</td>
</tr>
</tbody>
</table>
Annular weight or surcharge weight

2.5 Kgs of surcharge wt of 147mm dia are placed on specimen both at the soaking and testing of prepared samples.

Procedure

CBR test may be performed on undisturbed soil specimens.

- About 5kgs of soil is taken passing though 20mm IS sieve and retained on 4.75mm IS sieve and the soil is mixed with water up to OMC.
- The spacer disc is placed at the bottom of the mould over the base plate & a coarse filter paper is placed over the spacer disc.
- Then the moist soil sample is to be compacted over this in the mould by adopting either IS light compaction or IS heavy compaction.
- For IS heavy compaction 3 equal layers of compacted thickness about 44mm by applying 56 evenly distributed blows from 2.6kgs rammer.
- For IS heavy compaction 5 equal layers of compacted thickness about 26.5mm by applying 56 evenly distributed blows from 4.89 kg rammer.
- After compacting the last layer, the collar is removed and the excess soil above the top of the mould is evenly trimmed off by means of straight edge (of 5mm thickness).
- Clamps are removed and the mould with compacted soil is lifted leaving below the perforated base plate & the spacer disc which is removed and the mould with compacted soil is weighed
- Filter paper is placed on the perforated base plate & the mould with compacted soil is inverted & placed in position over the base plate and now the clamps of the base is tightened
- Another filter paper is placed on the placed on the top surface of the sample & the perforated plate with adjustable stem is placed over it.
- Now surcharge weights of 2.5 or 5kgs are placed over the perforated plate & the whole mould with the weights is placed in a water tank for soaking such that water can enter the specimen both from the top & bottom.
- The initial dial gauge readings are recorded & the test set up is kept undisturbed in the water tank to allow soaking of the soil specimen for full 4 days or 96 hrs.
- The final dial gauge reading is noted to measure the expansion & swelling of the specimen due to soaking.
- The swell measurement assembly is removed the mould is taken out of the water tank.
& the sample is allowed to drain in a perpendicular position for 15 min surcharge wt, perforated plate with stem, filter paper is removed.

- The mould with the soil subgrade is removed from the base plate & is weighed again to determine the wt of water absorbed.
- Then the specimen is clamped over base plate surcharge wt.’s is placed on specimens centrally such that the penetration test could be conducted. The mould with base plate is placed under the penetration plunger of loading machine.
- The penetration plunger is seated at the centre of the specimen & is brought in contact with the top surface of the soil sample by applying a seating load of 4kgs.
- The dial gauge for measuring the penetration values of the plunger is fitted in position
- The dial gauge of proving ring & the penetration dial gauge are set to 0.
- The load is applied through the penetration plunger at a uniform rate of 1.5mm/min
- The load reading are recorded at penetration reading 0, 0.5, 1.0, 1.5, 2, 2.5, 3, 4, 5, 7.5, 10 & 12.5mm.
- In case the load reading starts decreasing before 12.5mm penetration, the max load & the corresponding penetration values are recorded.
- After the final reading the load is released & the mould from loading machine.
- The proving ring calibration factor is noted so that load dial gauge value can be converted into the load in kg.

**Calculation**

Swelling or expansion ratio is calculated from the formula.

Expansion ratio = (100 (df – di))/h

Where,

df = Final dial gauge after soaking in mm

di = Initial dial gauge before soaking in mm

h = initial ht of the specimen in mm

\[
\text{CBR} = \frac{[\text{Load or pressure sustained by the specimen at 2.5 or 5.0mm penetration}]}{[\text{Load or pressure sustained by standard aggregates at the corresponding penetration level}]} \times 100
\]
Standard Load values on crushed stone aggregates for specified penetration values

<table>
<thead>
<tr>
<th>Penetration, mm</th>
<th>Standard Load, kg</th>
<th>Unit Standard Load, kg/cm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.5</td>
<td>1370</td>
<td>70</td>
</tr>
<tr>
<td>5.0</td>
<td>2055</td>
<td>105</td>
</tr>
</tbody>
</table>

- Generally, CBR value @ 2.5mm penetration is higher & this value is adopted.
- The initial upward concavity of the load penetration is due to the piston surface not being fully in contact with top of the specimen or when the top layer of soaked soil being too soft.

MODULUS OF SUBGRADE REACTION OF SOIL PLATE BEARING TEST

The plate bearing test has been devised to evaluate the supporting power of subgrade or any other pavement layer by using plates of larger diameter. Plate bearing test was originally meant to find the modulus of subgrade reaction in the Westergards’s analysis for wheel load stresses in cement concrete pavement. In the plate bearing test a compressive stress is applied to the soil or pavement layer through rigid plates of relatively large size & the deflection is measurement for various stress values. The deflection level is generally limited to a low value of 1.25mm to 5mm.

![Plate Bearing Test Set Up](image)

Modulus subgrade reaction (k)

- K may be defined as the pressure sustained per unit deformation of subgrade at specified pressure level using specified plate size.
- The standard plate size for finding K value is 75cm dia in same test a smaller plate of 30cm dia is also used (75,60,45,30 & 22.5 cm dia).

Apparatus used Bearing plate

Mild steel of 75cm dia & 1.5 to 2.5 cm thickness.
Loading equipment:
Reaction frame or dead load applied may be measured either by a proving ring or dial gauge assembly.

Settle measurement
It may be made by means of 3 or 4 dial gauge fixed on the periphery of the bearing plate from an independent datum frame. Datum frame should be supported from the loaded area.

Procedure
- At the test site, about 20cm top soil is removed & the site is levelled & the plate is properly seated on the prepared surface.
- The stiffening plates of decreasing dia are placed & the jack & proving ring assembly are fitted to provide reaction against the frame.
- 3 or 4 dial gauges are fixed on the periphery of the plate from the independent datum frame for measuring settlement.
- A seating load of 0.07 kg/cm² (320kgs for 75 dia) is applied & released after a few sec.
- The settlement dial gauges reading are now noted corresponding to zero load.
- A load is applied by means of jack sufficient to cause an average settlement of about 0.25mm.
- When there is no perception increase in settlement or when the rate of settlement is less than 0.025mm/min (case of clayey soil or wet soil), the reading of the settlement dial gauge is noted & the avg settlement is found & the load is noted from the proving ring dial reading.
- The load is then increased till settlement increases to a further amount of about 0.25mm & the avg settlement & load are found.
- The procedure is repeated till the settlement reaches 0.175cm.
- A graph is plotted with mean settlement versus mean bearing pressure (load/unit area) as shown in fig.

Bearing pressure settlement curve.
The pressure p (kg/cm²) corresponding to a settlement delta = 0.125cm (obtained from the graph shown above)
The modulus of subgrade reaction k is calculated from the relation given in kg/cm³
Correction for smaller plate size

In some cases, the load capacity may not be adequate to cause 75cm dia plate to settle 0.175cm. In such a case a plate of smaller dia (say 30cm) 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/cm²), the theoretical relationship of deformation (cm) under a rigid plate of radius a (cm) is given by

\[ \Delta = \frac{1.18 \cdot (p \times a)}{E} \]

From plate load test we know that, \( K = \frac{p}{\Delta} \)

Therefore

\[ K = \frac{p \times E}{1.18 \cdot (p \times a)} \]

\[ K = \frac{E}{1.18 \cdot a} \]

If the value of E is taken as constant for a soil, then \( k \times a = \) constant

I.e \( K \times a = k \times a_1 \) or \( k \cdot 30 \cdot a_{30} = k \cdot 75 \cdot a_{75} \)

Hence if the test is carried out with a smaller plate of radius a & the modulus of subgrade reaction K is found.
Then the corrected value of modulus of subgrade reaction $K$ for std plate of radius $a$, is given by

$$k_{75} = \frac{k_{30} \times a_{30}}{a_{75}}$$

Allowance for Worst Subgrade Moisture

The modulus of subgrade reaction $K$ of the soil will be lowest at the soaked condition. The moisture content at the time of carrying out plate load test may not represent the worst moisture condition and hence in such cases the value of modulus of subgrade reaction $K$ is found out the prevailing moisture content and the value so obtained is modified by applying a correction factor

$$k_s = k_u \frac{p_s}{p} = k_u \left( \frac{\Delta U}{\Delta S} \right)$$

Correction for Deflection or Bending of Mild Steel Plate

Given by IS 9214 -1980

$$k_b = 0.9556 \times K - 0.0113 \times K^2$$
6.3.1 Functions as Pavement Materials

Stone aggregates form the major portion of pavement structure and they form the prime materials used in the construction of different pavement layers. Aggregates used in various pavement layers have to bear different magnitudes of stresses due to the wheel loads. The aggregates of the pavement surface course have to resist: (i) the wear due to abrasive action of traffic (ii) deterioration due to weathering and (iii) the highest magnitude of wheel load stresses.

The stone aggregates are used in the construction of various pavement layers such as, (i) bituminous pavement layers of flexible pavements (ii) cement concrete mixes used for CC pavement slab and also for other cross drainage structures (iii) granular base course (iv) granular sub-base course or lean cement concrete sub-base and (v) drainage layer. Thus, stone aggregates form one of the important components of highway materials and therefore the properties of the aggregates are of considerable significance to the highway engineers.

Most of the road aggregates are prepared by crushing the natural rock. Gravel aggregates are small rounded stones of different sizes which are generally obtained as such from some river beds. Sand due to weathering of rock obtained from river beds is used as fine aggregate. ‘Manufactured sand’ obtained by crushing of hard rock are also made use of as fine aggregates. 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 or aggregates.

The aggregates are specified based on their grain size, shape, texture and its gradation. The crushed aggregates of different size are separated by sieving through square sieves of successively decreasing sizes. The required aggregate sizes are chosen to fulfil the desired gradation. The grading, tests and specifications of stone aggregates for different road making purposes have been specified by various agencies like the IRC, BIS, ASTM and BSI.

**Hard and soft aggregates**

Based on the strength property, the coarse aggregates may be divided as ‘hard aggregates’ and ‘soft aggregates’. Generally for the wearing 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.

Soft aggregates such as moorum, kankar, laterite, brick aggregates and slag have been used in the lower layers of road pavement structure. In the case of low-volume roads soft aggregates and soil-aggregate mixes (as per details given in Chapter – 9) can be advantageously made use of. A different set of tests and specifications are adopted for soft aggregates.

6.3.2 Desirable Properties of Road Aggregates and Tests

**Desirable properties**

The aggregates used in the pavement layers are subjected to impact due to heavy moving wheel loads. Therefore the aggregates used in pavement layers should have resistance to impact or possess ‘toughness’ property.
The aggregates used in pavement surface course have to withstand the high magnitude of load stresses and wear and tear. Therefore the aggregates should have sufficient resistance to abrasion caused by traffic movements or should possess ‘hardness’ property. The aggregates should also have resistance from getting polished or smooth rapidly under traffic movement in order to prevent the pavement surface becoming too slippery particularly under wet surface condition, resulting in accidents due to skidding of high speed vehicles.

The aggregates should have resistance to crushing and be able to retain the strength characteristics during the service life and therefore should possess adequate ‘strength’. They should not disintegrate under adverse weather conditions including alternate wet-dry and freeze-thaw cycles or in other words the stones should have enough resistance to weathering action or should possess ‘durability’ property.

The presence of air voids or pores in stones will result in lower specific gravity and also indicate lower strength characteristics and durability of the stones. The quantum of voids in aggregates is assessed by water absorption test. Higher values of water absorption in coarse aggregates are not desirable for use in bituminous mixes.

The fraction of aggregates which happen to fall in a particular size range, may have varying shapes and as a result may not have the same resistance to crushing and durability when compared with cubical, angular or rounded particles of the same stone. Too flaky and elongated aggregates should be avoided as far as possible as they can get crushed under the roller during compaction and also may break down under heavy wheel loads. Therefore angular shaped coarse aggregates are preferred in flexible pavement layers. The shape factor of coarse aggregates are defined in terms of flakiness index, elongation index and angularity number.

Affinity of aggregates to bituminous binders is an important property of coarse aggregates for use in the bituminous pavement layers. The chemical properties and the surface chemistry of the aggregate particles play important role in performance of bituminous pavements. In case the bituminous mix or the pavement layer is in contact with water for prolonged periods, stripping of bituminous binder is likely to take place from the coated aggregates, if the aggregates do not have affinity to bituminous binder.

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

(a) Resistance to impact or toughness,
(b) Resistance to abrasion or hardness,
(c) Resistance from getting polished or smooth/slippery,
(d) Resistance to crushing or crushing strength
(e) Good shape factors to avoid too flaky and elongated particles of coarse aggregates,
(f) Resistance to weathering or durability,
(g) Good adhesion or affinity with bituminous materials in presence of water or less stripping of bitumen coating from the aggregates.

Tests on road aggregates

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

(a) Aggregate impact test (to assess the toughness or resistance to impact)
(b) Los Angeles abrasion test (to evaluate the hardness and also toughness).
(c) Polished stone value test or accelerated polishing test.
(d) Aggregate crushing test (strength characteristics).
(e) Shape tests – flakiness index, elongation index and angularity number.
(f) Soundness test or durability test or accelerated weathering test.
(g) Specific gravity test and water absorption test.
(h) Bitumen adhesion test or stripping value test of aggregates.

All the above mentioned properties of aggregates and tests need not necessarily be conducted; the tests may be decided based on the type of pavement, the pavement layer, importance of the road and location including climatic factors. Some of the important properties and tests that are conducted on road aggregates are given here.

6.3.3 Aggregate Impact Test

During the construction process of pavement layers, particularly compaction by heavy rollers and also due to movement of heavy wheel loads of traffic, the road aggregates are subjected to impact or pounding action and there is possibility of some stones breaking into smaller pieces. The stone aggregates should therefore be sufficiently tough to resist fracture under impact loads. This property could differ from the resistance to crushing of aggregates under gradually increasing compressive stress.

The aggregate impact test is carried out to evaluate the resistance to impact of aggregates to fracture under repeated impacts; the test has been standardised by Bureau of Indian Standards (BIS).

The aggregate impact testing machine consists of a metal base and a cylindrical steel cup of internal diameter 102 mm and depth 50 mm in which the aggregate specimen is placed. A cylindrical metal hammer of weight 13.5 to 14.0 kg having a free fall from a height 380 mm is arranged to drop through vertical guides. The aggregate impact testing machine is shown in Fig. 6.15.

![Aggregate Impact Testing Machine](image)

**Fig. 6.15 Aggregate impact testing machine**
Aggregate specimen passing 12.5 mm sieve and retained on 10 mm sieve is filled in the cylindrical measure in three layers by tamping each layer by 25 blows by the tamping rod. The sample is weighed and 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 380 mm 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 fines passing 2.36 mm sieve formed in terms of the total weight of the sample.

The above test is repeated using another specimen of the same aggregate sample, by taking the same weight as in the first test. The mean of the two test results is reported as the Aggregate Impact Value (AIV) of the specimen, to the nearest whole number.

Based on the test results, the toughness property of the aggregate may be reported as given below:

<table>
<thead>
<tr>
<th>Aggregate impact value, %</th>
<th>Toughness property</th>
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</thead>
<tbody>
<tr>
<td>Less than 10</td>
<td>Exceptionally tough / strong</td>
</tr>
<tr>
<td>10 to 20</td>
<td>Very tough / strong</td>
</tr>
<tr>
<td>20 to 30</td>
<td>Good for pavement surface course</td>
</tr>
<tr>
<td>Above 35</td>
<td>Weak for pavement surface</td>
</tr>
</tbody>
</table>

The main advantage of aggregate impact test is that test equipment and the test procedure are quite simple; the test can be performed in a short time even at construction site or at stone quarry, as the apparatus is portable. Another advantage is that in addition to measuring the toughness value the test result is considered to give an indirect indication of the strength characteristics.

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. The Ministry of Road Transport and Highways (MORTH), Government of India has specified that the AIV of coarse aggregates used in Dense Bituminous Macadam (DBM) binder course and Semi-dense Bituminous Concrete (SDBC) surfacing should not exceed 27 percent and that used in Bituminous Concrete (BC) surface course should not exceed 24 percent.

### 6.3.4 Los Angeles Abrasion Test

**Various abrasion tests on road aggregates**

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

Abrasion test on aggregates are generally carried out by any one of the following methods:
(i) Los Angeles abrasion test
(ii) Deval abrasion test
(iii) Dorry abrasion test

Of these tests, the Los Angeles abrasion test is more commonly adopted as the test values of aggregates have been correlated with pavement performance studies. The Bureau of Indian Standards (BIS) has suggested that wherever possible, Los Angeles abrasion test method should be preferred over the Deval abrasion test. While specifying the minimum required resistance to abrasion of coarse aggregates to be used in different types of pavement layers, both the Indian Roads Congress (IRC) and the MORTH have specified Los Angeles abrasion test values only. Therefore in order to test the hardness property or resistance to abrasion of the coarse aggregates, only Los Angeles Abrasion test has been presented in this chapter.

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 aggregates and steel balls used as abrasive charge. During Los Angeles abrasion test, both abrasion or rubbing action between the aggregates and the steel balls and also impact or pounding action of these balls on the aggregates takes place. Therefore Los Angeles abrasion test is considered to be more reliable for evaluating the suitability of coarse aggregates in pavements as both abrasion and impact occur during the test similar to the field conditions. This test has been standardised by the BIS, ASTM and AASHTO. Acceptable limits of Los Angeles abrasion values of coarse aggregates have been specified by the IRC and also the MORTH.

The Los Angeles machine consists of a hollow cylinder closed at both ends, having inside diameter 700 mm and length 500 mm and mounted so as to rotate about its horizontal axis. A removable steel shelf projecting radially 88 mm into the cylinder and extending to the full length of it is mounted on the interior surface of the cylinder rigidly parallel to the axis. The abrasive charge consisting of cast iron spheres of approximate diameter 48 mm and each of weight 390 to 445 g is placed in the machine. The number of spheres to be used as abrasive charge and their total weight have been specified based on grading of the selected aggregate sample. Los Angeles abrasion testing machine is shown in Fig. 6.16.

The BIS has specified seven sets of grading of coarse aggregates, namely grading A, B, C, D, E, F and G; for each grading different weights of aggregate specimen and abrasive charge have been specified. For grading – A, total 5.0 kg of coarse aggregates consisting of 1250 g each of size ranges, (i) 40 to 25 mm (ii) 25 to 20 mm (iii) 20 to 12.5 mm and (iv) 12.5 to 10 mm are placed in the machine along with abrasive charge consisting of 12 spheres of total weight (5000 g +/− 25 g). For grading – B, total 5.0 kg of coarse aggregates consisting of 2500 g each of the coarse aggregates of size ranges, (i) 20 to 12.5 mm and (ii) 12.5 to 10 mm are placed in the machine along with abrasive charge consisting of 11 spheres of total weight (4584 g +/− 25 g).

The specified weight of aggregate specimen of desired grading is taken, (5 to 10 kg depending on gradation) and placed in the machine along with the specified 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 in terms of the original weight of the sample.
Fig. 6.16 Los angles abrasion testing machine

The Los Angeles abrasion value of good aggregates acceptable for bituminous concrete and other high quality pavement materials should be less than 30 percent; for cement concrete pavement and dense bituminous Macadam (DBM) binder course the maximum acceptable value is 35 percent; values up to 40 percent are allowed in granular base courses (like wet-mix Macadam and water bound Macadam) and in bituminous layers such as bituminous Macadam, bituminous carpet and surface dressing.

6.3.5 Polished Stone Value Test

Importance of the test

The aggregates used in the surface course of pavements are subjected to abrasion and rubbing action due to traffic movements and also during application of brakes. The presence of fine particles of sand and dust between the pavement surface and the tyres of vehicles accelerates the process of the pavement surface getting smoothened along the wheel paths. The smoothened pavement surface becomes slippery under wet conditions, resulting in skidding of high speed vehicles when brakes are applied suddenly and the wheels are locked. Therefore the aggregates used in pavement surface course of high speed highways should have resistance from getting polished or smooth rapidly under traffic movement in order to prevent the pavement surface becoming too slippery resulting in accidents due to skidding of high speed vehicles under wet weather condition.

Principle of the test

The ‘Polished Stone Value Test’ or the ‘Accelerated Polishing Test’ on aggregates is conducted in two stages. In the first stage the sample of stone aggregates is placed in a mould and subjected to accelerated polishing action in a machine under standard set of test conditions. In the second stage, the polished sample is subjected to friction test using a pendulum type skid resistance tester to determine the coefficient of friction expressed as a percentage, termed as the ‘polished stone value’.

Accelerated polishing

The test specimens are clamped around the rim of the road wheel which can be subjected to accelerated polishing test. The rubber tyred test wheel is lowered until it rests on the surface of the test specimens fixed around the road wheel. The specified weight is added at the end of the lever and the road wheel rotated at a speed of 320 r
325 rpm. Abrading materials and water are released at the specified rate and these are uniformly spread over the surface of the test specimen and the tyre of the test wheel where they are in contact. As the road wheel is rotated, the test specimens are subjected to abrading action or polishing for specified period of time. The machine is stopped and the test specimens are thoroughly cleaned by washing with water to remove sand and other fine particles of stone. The polished set of specimens is now ready for determination of the friction coefficient/skid number or the Polished Stone Value.

**Friction coefficient test**

The coefficient of friction or the skid resistance value of the test specimen is determined using pendulum type friction tester. The sample of polished specimen is fixed under the sliding portion of the rubber shoe of the pendulum head so as to test the friction coefficient of the sample. The height of the pendulum hinge is adjusted and fixed such that the sliding length of the rubber shoe is 75 mm. The surfaces of the specimen and the rubber shoe are wetted with water.

The pendulum and the pointer are released from the horizontal position and the pointer reading is noted as the ‘Skid Number’ or the ‘Polished Stone Value’ from the graduated scale and is recorded. The mean of the two values of the skid number or coefficient of friction expressed as percentage, is reported as the Polished Stone Value of the stone aggregate, to the nearest whole number.

As per the MORTH Specifications, the Polished Stone Value of coarse aggregates used in Bituminous Concrete and Semi Dense Bituminous Concrete surfacing of roads should be not less than 55.

**6.3.6 Aggregate Crushing Value Test**

The stone aggregates used for the construction of road pavements should possess satisfactory resistance to crushing under the roller during construction and under the application of heavy wheel loads on the pavement during its service life.

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. Aggregates possessing high resistance to crushing or low aggregate crushing value are preferred for use in high quality pavements.

The apparatus for the standard test consists of a steel cylinder 152 mm diameter with a base plate and a plunger, compression testing machines, cylindrical measure of diameter 115 mm and height 180 mm, tamping rod and sieves.

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 W1 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 W2 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.

\[
\text{Aggregate crushing value} = \frac{100W_2}{W_1} \text{ percent}
\]
The mean of the crushing value obtained in the two tests is reported as the aggregate crushing value, correct to the first decimal place.

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. The IRC and BIS have specified that the aggregate crushing value of the coarse aggregates to be used for cement concrete pavement surface should not exceed 30 percent. However aggregate crushing values have not been specified by the IRC or the Ministry of Road Transport and Highways for coarse aggregates to be used in flexible pavement/bituminous pavement construction methods.

6.3.7 Shape Tests

Importance of shape of coarse aggregates

The shape of aggregate particles is determined by the percentage of flaky and elongated particles contained in it. In the case of gravel, the shape may be expressed in terms of the angularity number. Presence of flaky and elongated particles in the coarse aggregates used for the construction of base and surface courses of road pavements is considered undesirable, as these may cause inherent weakness with possibilities of breaking down during compaction as well as under heavy traffic loads. Angular shapes of particles are desirable for granular base course and also for use in bituminous mixes due to increased stability derived from the better interlocking. When the shape of aggregates deviates more from the spherical shape, as in the case of angular aggregates, the void content in an aggregate of any specified size increases and hence the grain size distribution of graded aggregate has to be suitably altered in order to obtain minimum voids or the highest dry density in the dry mix.

The evaluation of shape of the particles is made in terms of flakiness index, elongation index and angularity number.

Flakiness index

Flakiness index (FI) of aggregate is the percentage by weight of aggregate particles the least dimension/thickness of which is less than three fifths or 0.6 of their mean dimension. This test is applicable to sizes larger than 6.3 mm. Standard thickness gauge is used to gauge the thickness or least dimension of the aggregate samples (see Fig. 6.17). The flaky aggregates are those which pass through the designated slots of the thickness gauge which has elongated slots with least dimension equal to 0.6 times of the mean dimension of each size range; these flaky aggregates are separated.

Fig. 6.17 Thickness gauge
The sample of aggregates to be tested is first 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 selected size range of aggregate in a group is 20 - 16 mm (i.e., passing 20 mm and retained on 16 mm sieve), 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 aggregates are added up and let this total weight of flaky particles be $W_1$ g. If the total weight of sample taken from the different size ranges is $W$ g, the flakiness index is given by $(100W_1)/W$ percent; in other words FI is the percentage of flaky materials, the widths of which are less than 0.6 of the mean dimensions.

The IRC has suggested that the FI of aggregates used in bituminous concrete and surface dressing should not exceed 25 %; the aggregates used in water bound Macadam and bituminous Macadam should not exceed 15 %.

**Elongation index**

Elongation index (EI) of an aggregate is the percentage by weight of particles, the greatest dimension of which or its length is greater than one and four fifth or 1.8 times their mean dimension. The elongation index test is not applicable for sizes smaller than 6.3 mm. Standard length gauge is used to gauge the greatest dimension or length of the aggregate samples (see Fig. 6.18). The elongated aggregates are those which do not pass through the designated slots of the length gauge which are 1.8 times of the respective mean size of the aggregate; these elongated pieces of aggregates are separated.

![Fig. 6.18 Length gauge](image)

The sample of aggregate to be tested is sieved through a set of sieves and separated into specified size ranges. The longest side of aggregate particles from each of the size range is then individually passed through the appropriate gauge of the length gauge; 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 size range is weighed. The total weight of the elongated stones is expressed as a percentage of the total weight of the sample taken to obtain 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.

**Determination of combined flakiness and elongation index**

The Ministry of Road Transport and Highways (MORTH), Government of India has specified the permissible limit of the combined flakiness and elongation index or
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combined index (CI) for coarse aggregates to be used in different types of pavement layers. MORTH has suggested that the flakiness index test should be carried out on the selected sample of coarse aggregates, as per the Bureau of Indian Standards (BIS) given in the above paragraphs and the value of the flakiness index, FI is determined.

The flaky particles passing through the respective slots of the thickness gauge are removed and the elongation index test is carried out on the remaining non-flaky particles. Let the value of elongation index so determined be EI. The combined index, CI of the coarse aggregate sample is then equal to (FI + EI).

MORTH has specified the maximum permissible value of the combined index of coarse aggregates as 30 % for wet mix Macadam base course, dense bituminous Macadam binder course and bituminous concrete surface course.

Angularity number

Based on the shape, the aggregate particles may be classified as rounded, partly rounded, irregular, angular or 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 interlocking property of compacted aggregate layer and also 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. A well compacted single sized rounded aggregates is found to have a solid volume of 67 percent and void volume of 33 percent. The angularity number of aggregate expressed in terms of the voids in a sample of single sized aggregates compacted in a particular manner. Angularity number is defined as (67 – percent solid volume of aggregates). The solid volume of the aggregate is found by filling it in a vessel in a specified manner. Thus, the angularity number measures the voids in excess of 33 percent. The higher the angularity number, more angular is the aggregate. The angularity number for aggregates used in constructions generally range from 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 W g. Then the cylinder is emptied and the weight of water filling the cylinder is determined = C g. The specific gravity $G_a$ of the aggregate is also determined.

The angularity number, AN is found from the formula:

$$AN = 67 - \frac{100W}{CG_a}$$

(Eq. 6.10)

This value is expressed as the nearest whole number.
6.3.8 Other Tests on Coarse Aggregates

There are a few other tests that are carried out for assessing the properties of coarse aggregates and their suitability for use in the construction of pavement layers. Of these the basic principle of the following tests are given in this chapter.

(a) Specific gravity and water absorption test
(b) Soundness test
(c) Stripping value test

Specific gravity and water absorption tests

**Significance of the tests**

The specific gravity of a stone aggregate is considered to be a measure of strength or quality of the material. Stones having low specific gravity are generally weaker than those with higher specific gravity values. The specific gravity tests helps in identification of stone. The specific gravity value of aggregates is made use of for making weight-volume conversions and for calculating the void content in compacted bituminous mixes.

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

**Determination of specific gravity and water absorption**

About two kg of dry sample of coarse aggregate 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, surface dried well with absorbent cloth and weighed. The aggregates are then dried in an oven at a temperature 110°C for 24 hours and then the oven 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 oven dried weight of the aggregates.

The specific gravity value of rocks generally varies 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 other-wise found suitable.

**Soundness test**

Soundness test is intended to study the resistance of aggregates to weathering action by conducting ‘accelerated weathering test cycles’. 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, as per the BIS.

Clean, dry aggregate specimen of specified size range is weighed and the number of pieces counted. The aggregate sample 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 to 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 fraction of coarse aggregate in each size range is sieved through specified sieves.

As a general guidance, 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. However the IRC has specified the maximum permissible loss in weight after five wet–dry cycles as 12 percent with sodium sulphate and 18 percent for magnesium sulphate for aggregates to be used in the bituminous binder course and surface course of flexible pavements.

Stripping value of road aggregates

Adhesion of bituminous binder with aggregates

Bituminous binders adhere well to all normal types of aggregates provided they are dry and are not dusty. In the absence of water there is practically no adhesion problem in bituminous road construction. The process of coating the aggregates is controlled largely by the viscosity of the binder. When the viscosity of the binder is high, coating of aggregates by the binder is slower.

Two problems are observed due to the presence of water. First problem is that if aggregate is wet and cold, it is normally not possible to coat with a bituminous binder. The water film from the wet aggregates can be removed by heating the aggregates and mixing can be done at higher temperature. Second problem is ‘stripping’ or detachment of coated binder from the aggregate due to the presence of water. This problem of stripping is experienced when the bituminous pavement layer is subjected to prolonged soaking under water and the problem is more predominant in bituminous mixes which are permeable to water and when certain types of aggregates are used in bituminous construction work.

The stripping is found be more in certain types of aggregates due to the fact that these aggregates have greater affinity towards water than with bituminous binders; the displacement of bituminous coating from the aggregates depends on the physico-chemical forces acting on the system.

Assessment of suitability of aggregates with respect to adhesion

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

The static immersion test is very commonly used as it is easy and simple. The principle of this type of test is immersing aggregates coated with binder in water and estimating the degree of stripping of the bituminous binder. A stripping test method has been developed by the Road Research laboratory (RRL) England. The method of test for assessing the stripping value of coarse aggregates coated with bitumen has been standardised by the BIS and this test has been briefly presented in this chapter.
Stripping value test

200 g of dry and clean stone aggregate passing 20 mm IS sieve and retained on 12.5 mm sieve is heated up to 150°C. The heated aggregate is mixed with five percent by weight of bitumen binder heated to 160°C. The aggregate and binder are mixed thoroughly till they are completely coated and mixture is transferred to a 500 ml beaker and allowed to cool at room temperature for about two hours. Distilled water is then added to immerse the coated aggregates. The beaker is covered and kept in a water-bath maintained at 40°C, taking care that the level of water in the water-bath is at least half the height of the beaker. After 24 hours, the beaker is taken out, cooled at room temperature and the extent of stripping from the individual aggregates is estimated visually.

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

The visual assessment of stripping value is more subjective and may lead to poor reproducibility. But still the test is an indicator as how a mixture of aggregates and binder may behave in the presence of water. Thus, an adhesion test such as the simple static immersion test or the stripping test would be suitable to assess whether the binder would adhere to the aggregate when immersed in water.

The IRC has specified the maximum stripping value as 25 percent for aggregates to be used in bituminous construction like surface dressing, bituminous Macadam and bitumen mastic. The maximum stripping value suggested by IRC is 10% for aggregates used in open graded premix carpet.

Alternate test method

The degree of bitumen adhesion may also be mechanically measured indirectly by measuring the change in a mechanical property of the compacted bituminous mix, such as compressive strength or any other strength test due to soaking under water. The percentage reduction in strength is an indicator of the extent of damage due to immersing the specimen of bituminous mix in the water.

Methods of dealing with problem of stripping

Most stone aggregates surfaces are electrically charged. As an example, silica a common constituent of igneous rocks possesses 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 electro-negative have greater affinity with water 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.

If the stripping value exceeds the specified value, use of anti-stripping agents may be recommended. Several anti-stripping agents are available, which when used with the bituminous mix could reduce the stripping.

Now bituminous binders are also available that are either 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.
6.4.1 Types and Characteristics of Bituminous Binders

Bituminous binders used in pavement construction works are (i) bitumen and (ii) tar. Bitumen is a petroleum product obtained by the distillation of petroleum crude. Coal tar is produced from coal as a by product of coke. Both bitumens and tar have similar appearance as both are black in colour. Though both these binders were used for pavement works, they have widely different characteristics. Tar is no longer used for paving applications because of its undesirable characteristics including high temperature susceptibility and harmful effects of its fumes during heating.

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 water proofing properties. The different grades of bitumen used for pavement construction work of roads and airfields are called paving grade bitumen and those used for water proofing of structures and industrial floors etc. are called industrial grade bitumen.

Paving grade bitumen which is obtained from the distillation process of petroleum crude is extensively used in the construction of flexible pavement layers, particularly the surface and binder courses. At normal range of atmospheric temperature, bitumen is in semi-solid state and remains highly viscous and sticky. When the paving grade bitumen is heated, it softens at a rapid rate and attains fluid consistency and the viscosity decreases with further increase in temperature. For the construction of bituminous pavements, the paving grade bitumen is heated to temperatures in the range of 130 to 175 °C or even higher, depending upon the type and grade of bitumen selected and the type of the construction work. Mixing of the bitumen with the aggregates is done in a hot mix plant to obtain ‘hot bituminous mix’.

In order to achieve fluid consistency of the bitumen at relatively low temperatures with nominal heating, ‘cut-back bitumen’ has been developed. Cutback bitumen is prepared by diluting a paving grade bitumen with a volatile solvent such as a light fuel oil or kerosene. The consistency of the cut-back and the rate at which it hardens after application depends on the grade of the bitumen selected and the characteristics and proportion of the light oil/diluent used.

Another entirely different approach of achieving fluid consistency of bitumen for use in road works without the need to heat the binder is the ‘bitumen emulsion’. Bitumen emulsion or emulsified bitumen is prepared by dispersing bitumen in the form of fine globules suspended in water with the help of a suitable emulsifier. The properties of bituminous emulsions vary depending upon the properties of the bituminous binder, its proportion with respect to water and the properties of the emulsifier. Appropriate type and grade of bitumen emulsion may be selected for being directly sprayed as prime coat or tack coat and for being mixed with aggregates to prepare ‘cold bituminous mix’.

The viscosity of ordinary paving grade bitumen varies considerably with temperature, resulting in bituminous pavement surface course being susceptible to temperature changes. During hot weather, the bituminous pavement surface course becomes soft and during cold weather it becomes too stiff and brittle with the possibility of early cracking. Bitumen modifiers reduce the temperature susceptibility
of the bituminous binder and that of the bituminous mix. Bituminous mixes prepared using suitable type of modified binders offer better resistance to deformation at higher temperatures and remains relatively more flexible and elastic at low temperatures.

Thus the types of bituminous binders that are used in flexible pavement construction are:

(a) Paving grade bitumen
(b) Modified bituminous binders
(c) Cut-back bitumen and
(d) Bitumen emulsion

Of the above binders, the paving grade bitumen and modified bituminous binders need heating before being used in paving applications. Cut-back bitumen may or may not need slight heating depending on the selected grade of the binder and the site temperature during mixing. When bitumen emulsion is used in pavement construction, no heating is required. Bituminous emulsions are also available with modifiers.

6.4.2 Functions of the Binders as Pavement Material and Desirable Properties

Bituminous binders are very commonly used in surface course of pavements; they are also used in the binder and base courses of flexible pavements to withstand relatively adverse conditions of traffic and climate. Bituminous binders are used for preparation of bituminous mixes by mixing with selected aggregates, either in the form of hot bituminous mix or cold mix. Bituminous binders are also used in other techniques of construction such as, ‘surface dressing’ to be used as a thin surfacing course or in ‘penetration Macadam’ for use in the base course.

Bituminous binder is used in the form of bitumen emulsion, as a ‘prime coat’ over granular base course of flexible pavement. The binder in the form of emulsion is also used as a tack coat to be sprayed over the primed base course or over an existing bituminous surface, before laying a bituminous pavement layer.

The bituminous binder (in the form of cut-back or emulsion) may be used in soil-bitumen stabilisation. The bituminous binder may also be used for the preparation of sealer materials for filling the joints and cracks in cement concrete pavements.

The desirable properties of bitumen depend on the type of bituminous construction. In general the bitumen should possess the following desirable properties:

(a) The viscosity of the bitumen at the time of mixing with aggregates and compaction of the pre-mix should be adequate. This is achieved either by (i) heating the bitumen and aggregate prior to mixing or (ii) by using in the form of cut-back or (iii) by using in the form of emulsion of suitable grade.

(b) The bituminous binder should become sufficiently viscous on cooling (or on evaporation of the volatile solvent of the cut-back or the water of the emulsion) that the compacted bituminous pavement layer can gain stability and resist deformation under traffic loads.

(c) It is desirable that the bitumen binder used in the bituminous mixes form ductile thin films around the aggregates to serve as a satisfactory binder in improving the physical interlocking of the aggregates. The binder material which does not possess sufficient ductility would crack and thus provide pervious pavement surface.
(d) The bituminous binder used should not be highly temperature susceptible. During the hottest weather of the region the bituminous surface 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 also be durable to sustain adverse weathering effects.

(e) The bitumen binder should have sufficient adhesion with the aggregates in the mix in presence of water.

(f) There has to be adequate affinity and adhesion between the bitumen and aggregate used in the mix. The coated binder should not strip off from the stone aggregate under stagnant water.

6.4.3 Tests on Bitumen

Objects

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 the Bureau of Indian Standards (BIS), American Society for Testing and Materials (ASTM), Asphalt Institute and the British Standards Institution.

The common tests to assess the properties and requirements of paving grade bitumen are the viscosity tests, penetration test, ductility test and the softening point test. Also specific gravity test and flash and fire point tests are needed for use in paving applications. Additional tests like the matter soluble in carbon-disulphide, loss on heating and penetration test on residue may also be carried out.

Earlier the classification of bitumen was based on the penetration and ductility test results. It was later observed that bitumen from different sources possessing same penetration value at a specified temperature may exhibit entirely different viscosity characteristics and hence different temperature susceptibility characteristics at the application and service temperatures. Therefore it is important to determine the viscosity property of the binder in terms of ‘absolute viscosity’ and ‘kinematic viscosity’ test results.

Various tests that are generally carried out to evaluate the properties of bitumen binders are:

(a) Penetration test
(b) Viscosity tests
(c) Ductility test
(d) Softening point test
(e) Specific gravity test
(f) Flash and Fire point tests
(g) Loss on heating test
(h) Solubility test

These importance and principle of these tests are briefly given here.

Penetration test

The consistency of bituminous materials varies depending upon several factors such
as constituents, temperature, etc. At temperature ranges between 25 and 50°C most of the paving bitumen grades remain in semi-solid or in plastic state. Determination of absolute viscosity of bituminous materials is not so simple. Therefore the consistency of these materials is determined by indirect methods. Penetration test is one such indirect test to determine the consistency of paving grade bitumen, which is a very simple test.

The penetration test is widely used for classifying the bitumen into different grades. The BIS has standardized the penetration test equipment and the test procedure. The penetration test determines the consistency of these materials for the purpose of grading them by measuring the depth to which a standard needle will penetrate vertically under specified conditions of standard load, duration and temperature. Thus the basic principle of the penetration test is the measurement of the penetration (in units of one tenth of a mm) of a standard needle in a bitumen sample maintained at 25°C during five seconds, the total weight of the needle assembly being 100 g. The concept of the penetration test on bitumen sample is illustrated in Fig. 6.19.

![Penetration Test Diagram](image)

**Fig. 6.19 Concept of penetration test on bitumen**

Penetration test apparatus or the penetrometer consists of a penetration needle assembly which is attached to a calibrated dial. On release, the penetration needle penetrates into the bitumen specimen without appreciable friction. (See Fig. 6.20) 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, placed under the penetrometer and the needle is adjusted to make contact with the surface of the sample. The dial is set to zero or the initial reading is 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 at least 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 mm 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.

Penetration test is the most commonly adopted to determine the grade of the bitumen in terms of its hardness because of its simplicity. Softer the bitumen, the greater will be the penetration value. 80/100 bitumen denotes that the penetration value of the binder ranges between 80 and 100. The penetration grades of bitumen binders are generally denoted as 80/100, 60/70 or 30/40 grade bitumen.

Some of the limitations of penetration test for grading of bitumen binders are:
Fig. 6.20 Penetrometer

(i) penetration test is an empirical test and it has no relation with the fundamental properties of the binder (ii) the test temperature of 25°C is not the general pavement service temperature (iii) the service temperature of the pavement is much higher say, about 60°C for most part of the day in several regions (iv) bitumen having the same penetration value may have different performance while in service depending on its temperature susceptibility; this is because bitumen having the same penetration value may have widely varying temperature-stiffness relationship.

In view of the above limitations, grading of bituminous binders is done based on viscosity test results. ‘Viscosity Grading’ of bitumen has been recommended by the BIS for paving applications.

Viscosity Tests

Viscosity of bituminous binders

Viscosity of a liquid is the property that retards its flow due to internal friction and it is a measure of resistance to flow of the liquid. The flow of a liquid under an applied force will depend on its viscosity; higher the viscosity, slower will be its movement or rate of flow. The range of viscosity of different types of bituminous binders (such as hot bitumen, cutback bitumen or bitumen emulsion) used in road construction vary considerably depending on the type and grade of the binder and the temperature of application. Therefore different test methods are necessary for the determination of the viscosity of the bituminous binders in liquid state and the method chosen will depend upon the viscosity of the binder to be tested and the purpose for which the measurement is required.

A number of test methods and apparatus have been developed for testing of bituminous binders, some of these are empirical methods which give an indirect
measure of viscosity, making use of orifice type viscometers and others are for the
direct measurement of absolute viscosity. Various terms that are used to express
viscosity of bituminous binders are given below.

Absolute viscosity

The ratio between the applied shear stress and the rate of shear is called the
coefficient of viscosity or the 'Absolute Viscosity' of the liquid. Absolute or dynamic
viscosity (of a Newtonian liquid, in which the shear stress is directly proportional to the
rate of shear strain) is the internal friction such that a tangential force of one dyne (or
0.00001 N) acting on planes of unit area separated by unit distance of liquid produces
unit tangential velocity. In CGS units the viscosity is measured as gram per cm-second
(g/cm-s) or dyne-s/cm² and is termed, Poise (P). The SI unit of viscosity is Pascal-
second (Pa-s) or Newton – second per square metre (N-s / m²) and is equal to 10 P.

Kinematic viscosity

Kinematic Viscosity (of a Newtonian liquid) is the ratio of the absolute viscosity to
the density of the liquid, both at the same temperature. It is a measure of resistance to
flow of a liquid under gravity. The CGS unit used for the measurement of kinematic
viscosity is cm²/second and is called a Stoke (St). In SI units, kinematic viscosity is
expressed in units of mm²/second or in centi-stoke, cSt which is one hundredth of a
stoke, i.e., 1 mm²/second = 1 cSt

If kinematic viscosity (in stokes) is multiplied by the specific gravity of bitumen,
the absolute viscosity (in poise) can be obtained.

Indirect measurement of viscosity

Viscosity is indirectly measured by determining the time taken by 50 ml of the
binder in fluid state to flow through a specified orifice from a cup, under standard test
conditions and specified temperature. This method is suitable for measuring viscosity
of bitumen emulsion, cut-back bitumen and tar.

Measurement of viscosity of bituminous binders

Some of the important methods of measuring absolute viscosity of bitumen are:

(i) simple shear of a thin film placed between two parallel flat plates, such as the
sliding plate viscometer,

(ii) shear between rotating coaxial cylinders or cone and cylinder, such coni-
cylindrical viscometer or Brookfield viscometer and,

(iii) flow through capillary tube, such as vacuum capillary viscometer.

Equipment like sliding plate micro viscometer and Brookfield viscometer are in use
for measurement of viscosity of bitumen of all grades irrespective of testing temperature.
The viscosity of bitumen can also be measured by a capillary tube viscometer.

In this chapter, the following methods for determination of viscosity of bituminous
binders have been briefly presented:

(a) Absolute Viscosity of paving grade bitumen using vacuum capillary tube
viscometer,

(b) Kinematic Viscosity of bitumen and cutback bitumen using capillary type
viscometer.

(c) Indirect measurement of viscosity of bituminous emulsion and tar by using
orifice viscometers.
**Determination of absolute viscosity by vacuum capillary viscometer**

A vacuum capillary tube viscometer is generally used to measure the absolute viscosity of bitumen at 60 °C. The viscometer is mounted in a thermostatically controlled water bath or oil bath at uniform test temperature of 60 °C. At this temperature the paving grade bitumen is highly viscous and cannot flow freely through the capillary tube and therefore there is a need to apply vacuum pressure. The time taken (in seconds) for the liquid bitumen to flow through a known distance in a capillary tube is measured and expressed as the viscosity. Depending on the type of fluid, different diameter tubes are to be used and hence the calibration factors supplied by the manufacturer are necessary.

The measured time, t (sec) is multiplied by the calibration factor C of the viscometer in centi-Stokes per second (cSt/sec) to obtain the value of kinematic viscosity in centi-Stokes.

Kinematic viscosity (at test temperature of 135°C / 60°C), cSt = Ct

BIS has standardised three types of ‘Vacuum Capillary Viscometers’ for the determination of absolute viscosity of bitumen, namely ‘Cannon-Manning’, ‘Asphalt Institute’ and ‘Modified Coppers’ Vacuum Capillary Viscometer’.

**Determination of viscosity using orifice viscometer**

Viscosity of liquid bituminous binders like bitumen emulsion and tar are determined by indirect method using orifice type viscometers. A specified quantity of the binder (50 ml) is allowed to flow through specified orifice size of the test-cup at a given temperature and the time taken in seconds is recorded as the viscosity value. The test concept is illustrated in Fig. 6.21. As per the specifications of Bureau of Indian Standards, the viscosity values of bitumen emulsions are determined using ‘Saybolt Furol’ orifice viscometer at test temperatures of 25 °C and 50 °C. The viscosity values of tar are determined using orifice viscometer called ‘Tar Viscometer’ using either 10 mm or 4 mm size orifice.

![Concept of test using orifice type viscometer](image)

**Fig. 6.21 Concept of test using orifice type viscometer**

**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. The ductile film of binder improves 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 permit the surface water to enter into the pavement resulting in rapid deterioration and failure. Ductility test is carried out on bitumen to test 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.

The ductility value is expressed as the distance in centimetre (cm) to which the bitumen specimen of standard size can be stretched before the thread breaks. The standard briquette specimen has a minimum cross section 10 mm x 10 mm. The test is conducted at 27°C with a rate of pull of 50 mm per minute, until the stretched specimen breaks. The ductility test concept is shown in Fig. 6.22.

![Ductility Test Concept](image)

**Fig. 6.22 Ductility test concept**

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 ductility test specimen and mould are shown in Fig. 6.23. The samples along with the moulds are cooled in air and then in water bath maintained at 27°C. The excess bitumen material is trimmed and the surface is levelled 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 to the machine and the pointer is adjusted to zero. The distance up to the point of breaking of thread is reported as ductility value, in cm. 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 specimen briquettes, test temperature and rate of pulling.

![Ductility Test Specimen and Mould](image)

**Fig. 6.23 Ductility test specimen and mould**
The ductility values of bitumen generally vary from 5 to over 100 for different bitumen grades. A minimum ductility value of 50 to 75 cm is generally specified for bitumen used in pavement construction.

Ductility values have also been specified on residue obtained after conducting 'thin film oven test' (TFOT) on bitumen binder. The BIS has specified that the ductility values on residue from TFOT of paving bitumen of viscosity grades, VG-10, VG-20, VG-30 and VG-40 should not be less than 75, 50, 40 and 25 cm respectively.

**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 concept of softening point test and the test set-up is shown in Fig. 6.24. Generally higher softening point indicates lower temperature susceptibility and is preferred in warm climates.

![Softening point test set-up](image)

**Fig. 6.24 Softening point test set-up**

A brass ring containing test sample of bitumen is suspended in a beaker with liquid bath; water is used as the bath if the softening point is less than 80°C and glycerine is used for temperatures exceeding 80°C. 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 plate placed at a specified distance below the ring is recorded as the softening point of the bitumen. Harder grades of bitumen possess higher softening point than soft grade bitumen.

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

**Specific gravity test**

The specific gravity 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 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 binder is defined as the ratio of the mass of a given volume of the binder to the mass of an equal volume of water, the temperature
of both being at 27°C. The specific gravity is determined either by using a pyknometer or by preparing a specimen of cube shape in semi-solid or solid state and by weighing in air and water. The specific gravity is obtained by dividing the weight of the bitumen by weight of equal volume of 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.

**Flash and fire point tests**

When a bituminous binder is heated continuously, above a certain temperature it starts emitting volatile vapours and these volatile vapours can momentarily catch fire causing a flash, though the binder itself does not catch fire and burn at this temperature. The temperature at which such behaviour occurs is found to differ for different types and grades of bituminous binders. This condition is very hazardous and therefore it is essential to determine the temperature at which the flash of fire can occur in each type and grade of bituminous binder.

Flash point test gives an indication of the critical temperature at and above which suitable precautions should be taken while heating the binder. In order to eliminate fire hazards during heating, mixing or application, the paving engineers should restrict the mixing and application temperatures well below this temperature.

The ‘flash point’ of a bituminous binder is defined as the lowest temperature at which application of a test flame causes the vapours of the binder to catch an instant fire in the form of flash under specified test conditions. Flash point test concept is illustrated in Fig. 6.25. Two types of test apparatus may be used for conducting flash point test on bitumen, namely the Pensky-Martens Closed Cup Tester and Open Cup Tester.

![Flash Point Test Concept](image)

**Fig. 6.25 Flash point test concept**

If the bituminous binder is further heated to a temperature higher than the flash point, the binder material itself catches fire and continues to burn, the lowest temperature causing this condition is called the ‘fire point’. The fire point is always higher than the flash point of a material. The fire point is defined as the lowest temperature at which application of a test flame causes the binder material to ignite and burn at least for five seconds under specified test conditions. Pensky-Martens Open Cup Tester is made use of to determine the fire point of the bituminous binders.
Pensky-Martens closed cup apparatus or open cup are used for conducting the tests. The material to be tested is filled in the cup up to 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°C 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.

In order to determine the fire point, 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 BIS has specified the minimum value of flash point by open cup test as 220°C for all the grades of paving bitumen. The minimum specified value of flash point by Pensky-Martens closed cup test for rapid curing cutback bitumen (RC) is 26°C, for medium curing cutbacks, MC 30 and MC 70 is 38°C, for MC 250, MC 800 and MC 3000 is 65°C and for slow curing cutbacks (SC) the minimum specified flash point values range from 65 to 107°C.

**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 50 g of the sample is weighed and heated to a temperature of 163°C for 5 hours in a special oven designed for this test. After the heating period, this specimen is cooled and weighed again 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 up to 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.

**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 2.0 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; the insoluble material is expressed as a percentage by weight 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 2.0 g of bitumen is dissolved in 10 ml of naphtha. A drop of this solution is taken out and placed on a filter paper, the first drop after one hour and second one after 24 hours...
after the solution is prepared. If the stain of the spot on the filter paper is uniform in colour, the bitumen is accepted as not cracked. 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.

**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 specimen is determined by mixing a 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.

**6.4.4 Grading of Bitumen**

In India until recently bitumen binder for use in pavement construction was classified into various ‘penetration grades’ such as 80/100, 60/70, 30/40, etc. based on the penetration test values determined at 25°C. Now a more rational method of grading paving bitumen, known as ‘Viscosity Grading’ (VG) has been adopted by the Bureau of Indian Standards (BIS) for grading of bitumen in India, based on the absolute viscosity values determined at 60°C and kinematic viscosity values determined at 135°C. Generally pavement service temperature is considered to be around 60°C and the laying temperature of hot bituminous mixes to be about 135°C. Therefore viscosity grading system based on viscosity tests conducted on bitumen at these temperatures are considered more reliable than the grading method based on penetration test. However two similar viscosity grades of bitumen from different sources may have different viscosity values after exposing to Thin Film Oven Test (TFOT) and hence may behave differently during and after construction.

The four grades of bitumen currently adopted in India based on viscosity values and their respective penetration values at 25 °C are given in Table 6.7.

<table>
<thead>
<tr>
<th>Sl. no.</th>
<th>Viscosity grading</th>
<th>Absolute viscosity at 60°C, poise (min.)</th>
<th>Kinematic viscosity at 135°C, cSt (min.)</th>
<th>Range of penetration value at 25°C</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>VG 10</td>
<td>800</td>
<td>250</td>
<td>80 – 100</td>
</tr>
<tr>
<td>2</td>
<td>VG 20</td>
<td>1600</td>
<td>300</td>
<td>60 – 80</td>
</tr>
<tr>
<td>3</td>
<td>VG 30</td>
<td>2400</td>
<td>350</td>
<td>50 – 70</td>
</tr>
<tr>
<td>4</td>
<td>VG 40</td>
<td>3200</td>
<td>400</td>
<td>40 – 60</td>
</tr>
</tbody>
</table>

The viscosity grades of bitumen recommended for use in India for paving applications are given in Table 6.8.

**Table 6.8 Recommended viscosity grades of bitumen for use in India**

<table>
<thead>
<tr>
<th>Viscosity grade (VG)</th>
<th>General applications</th>
</tr>
</thead>
<tbody>
<tr>
<td>VG-40</td>
<td>Use in high stressed areas like intersections, toll plazas, truck terminals, truck lay-byes in lieu of 30/40 penetration grade</td>
</tr>
<tr>
<td>VG-30</td>
<td>Paving applications for most part of India, in lieu of 60/70 penetration grade of bitumen</td>
</tr>
<tr>
<td>VG-20</td>
<td>Paving applications in cold climatic conditions of North India and in high altitude regions</td>
</tr>
<tr>
<td>VG-10</td>
<td>Spraying applications; paving applications in cold regions in lieu of 80/100 penetration grade</td>
</tr>
</tbody>
</table>
BITUMEN EMULSION

Characteristics of bitumen emulsion

A bitumen emulsion is a liquid product in which a substantial amount of bitumen is suspended in a finely divided condition in an aqueous medium and stabilized by means of one or more suitable materials. An emulsion is a two phase system consisting of two immiscible liquids; the one being dispersed as fine globules in the other.

The paving bitumen is broken up into fine globules and kept in water. The average diameter of globules of bitumen is about 2 microns. A small proportion of an emulsifier (half to one percent by weight of emulsion) is used to facilitate 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 is soaps, surface active agents and colloidal powders.

Two common methods followed for the preparation of emulsion are the colloid mill method and the high-speed mixer method. The manufactured emulsion is stored in air tight drums. Bitumen emulsion shall be homogeneous and it should not show undispersed bitumen after thorough mixing within one year from the date of manufacture.

The bitumen emulsion may be of anionic type or cationic type. The choice of the type of emulsion for a particular situation depends on the aggregate type, climatic conditions, and environmental conditions. Five types of bitumen emulsions are prepared, namely: (i) Rapid Setting types, RS-1 and RS-2 (ii) Medium Setting type, MS and (iii) Slow Setting types, SS-1 and SS-2.

Tests on bitumen emulsions

The specified tests on bitumen emulsion are given below:

(a) Viscosity test – to assess ability to be sprayed through jets
(b) Water content – to estimate the actual binder quantity
(c) Settlement test – to evaluate settlement when left standing undisturbed
(d) Demulsibility test – to find the residue after mixing with calcium chloride as specified
(e) Miscibility in water – to assess coagulation due to addition of distilled water
(f) Cement mixing test – to assess stability in presence of fines in aggregates
(g) Coating test – to assess coating of stone aggregates
(h) Sieving test – to measure sedimentation of emulsion during storage
(i) Particle charge – to evaluate the type of charge

Uses and applications in road works

Bitumen emulsions have wide range of applications in road construction and maintenance works. The common examples in the construction of bituminous pavement layers are in the interface treatments as prime coat and tack coat and in various other works such as fog seal, seal coat, surface dressing, bituminous carpet, micro-surfacing, etc. The emulsions are extensively being used in maintenance works of bituminous pavements including the patch repair works, particularly during wet weather condition.

Department of Civil Engineering, MITE
When the bitumen emulsion is applied to the road surface, it breaks down and the binder starts coating the aggregates, though needed binding strength develops slowly as and when the water evaporates. The first sign of break-down of emulsion is shown by the change in colour of the film from chocolate brown of the emulsion to black colour of the binder.

The main advantage of bitumen emulsions are (a) they can be used, without heating for spraying or preparing mixes (b) they are particularly useful for patch repair works and can be used even when the surface is wet.

The rapid setting bitumen emulsions are used in spray applications like tack coat, for surface treatments, surface dressing and penetration Macadam.

The medium setting emulsion may be used in cold bituminous mixes in which the percentage of coarse aggregates are substantially high, with a desirable gradation of zero percent fines passing 75 micron sieve and they are also used for surface dressing and penetration Macadam.

The slow setting emulsions are used for prime coat, slurry seal treatments, recycling works and in soil stabilisation; they are also used with well graded bituminous mixes containing a substantial proportion of fine aggregates passing 2.36 mm sieve and a portion containing fines passing 75 micron sieve.

6.4.6 Cutback Bitumen

Characteristics and uses

Cutback bitumen is obtained by blending bitumen binder with suitable volatile diluents or solvents in the required proportion to reduce its viscosity to the desired range. After the cutback mix is used in construction work, the volatile solvent gets evaporated, the binder starts hardening and develops the binding properties. The rate at which the cutback hardens on the road depends upon the characteristics and quantity of the volatile oil used as the diluents and also on the atmospheric temperature and humidity at the work site.

Cutback bitumen binder of appropriate type and grade is selected for use as tack coat without the need to heat. This binder is particularly preferred for use in sites at sub-zero temperatures and in regions of high altitude. Cutback may also be used for preparing bituminous mixes and for soil-bitumen stabilization.

Types of cutback bitumen

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, which depends on the type and proportion of diluents/solvent used. Rapid curing cutback bitumen are classified by BIS, on the basis of initial kinematic viscosity into a four grades with designations RC-70, RC-250, RC-800 and RC-3000, in the increasing order of initial viscosity. RC-70 is rapid curing cutback of low initial viscosity to be sprayed at normal air temperature without heating, whereas the RC-800 and RC-3000 are products of high viscosity which cannot be easily mixed with fine aggregate or soil, at low temperatures.
Medium curing (MC) cutback bitumen is classified on the basis of initial viscosity into five grades: MC-30, MC-70, MC-250, MC-800 and MC-3000 in the increasing order of viscosity. MC-30 may be used as primer. Similarly the Slow Curing (SC) cutbacks are classified into four grades and are designated as: SC-70, SC-250, SC-800 and SC-3000.

Properties and tests

RC, MC and SC types of cutback bitumen of the various grades mentioned above should comply with the requirements with regard to the properties such as viscosity at different test temperatures, flash point, distillation fractions, residue from distillation up to the specified temperature and tests on residue from distillation. The following tests are generally carried out on cutback bitumen.

(i) Kinematic Viscosity
(ii) Flash point test (Penskey Marten’s closed type)
(iii) Distillation test (to find fractions of distillate up to 190, 225, 260, 315 and 360°C).
(iv) Tests on residue from distillation up to 360°C:
(v) Viscosity at 60°C
(vi) Ductility at 27°C
(vii) Matter soluble in Trichloro-ethylene
(viii) Water Content

6.4.7 Modified Bituminous Binders

Objects

The viscosity of ordinary paving grade bitumen varies considerably with temperature; as a result the bituminous pavement surface course also becomes susceptible to temperature changes. During hot weather the bituminous surface course becomes soft resulting in possibility of permanent deformation and early rutting along the wheel paths of heavy vehicles. During cold weather, the bituminous pavement surface course becomes too stiff and brittle with the possibility of early cracking under repeated application of heavy wheel loads.

Bitumen modifiers reduce the temperature susceptibility of the binder as well as that of the bituminous mix with consequent improvement in pavement stability by imparting visco-elastic properties to the mix. This product helps to reduce the permanent deformation or rutting of the bituminous surface course under traffic loads. Modified bituminous binders offer better resistance to deformation at higher temperatures and remains flexible and elastic at low temperatures.

The use of virgin polymers to modify the characteristics of the bituminous binder in bituminous mixtures is an accepted practice in the highway construction industry. Properties inherent in polymer additives enhance the performance characteristics of the bituminous binder and of the compacted bituminous pavement layer.

Characteristics

Some of the materials used as modifiers of the bitumen binder are: (a) Polymers – SBS (Styrene-Butadiene-Styrene), SBR (Styrene-Butadiene Rubber), EVA (Ethylene Vinyl Acetate) and (b) Rubber – Crumb rubber and Natural rubber. The advantages of
modified binders are improved resistance to cracking as stress or strain absorbing membrane (SAM) and stress absorbing membrane interface (SAMI). They provide reduced temperature susceptibility and more cohesive and tough binders to improve aggregate retention in high stress seal (HSS) applications.

Polymer modified binders (PMB) may be used with bituminous mixes to: (i) improve resistance to permanent deformation (ii) improve fatigue resistance and (iii) increase durability in bituminous mixes. Both elastomeric types and plastomeric types provide improved resistance to deformation as well as improved durability of open graded mixes.

Bituminous mixes produced with elastomeric PMB will generally have a lower stiffness modulus than those with conventional bitumen, but have a significantly high flexibility. Bituminous mixes with plastomeric PMB types may have the same or even higher stiffness than conventional bitumen, but without a large increase in flexibility. Therefore plastomeric type polymers should not be used where a high degree of flexibility is required during cold weather. Bituminous mixes prepared using crumb rubber (powdered scrap rubber) modified bitumen binders have also been used to provide improved flexibility, resistance to deformation and resistance to reflective cracking of bituminous pavement surface course.

Mixes using modified bituminous binders require special design considerations and care during construction operations.

**Classification**

As per BIS (vide IS 15462: 2004), polymer and rubber modified bitumen are classified into four types as given below:

(a) Type A - PMB (P) : Plastomeric thermoplastics based
(b) Type B - PMB (E) : Elastomeric thermoplastics based
(c) Type C - NRMB : Natural rubber and SBR latex based
(d) Type D - CRMB : Crumb rubber/treated crumb rubber based

Type A, Type B and Type C are further classified into three grades according to their penetration value. Type D is further classified into three grades according to softening point values. The grades of Type A - PMB (P) are as follows:

(i) PMB (P) 120- Type A - PMB (P) having a penetration value between 90 to 150.
(ii) PMB (P) 70- Type A - PMB (P) having penetration value between 50 to 90.
(iii) PMB (P) 40- Type A - PMB (P) having penetration value between 30 and 50.

Similarly the grades of Type B - PMB (E) are (i) PMB (E) 120, (ii) PMB (E) 70 and (iii) PMB (E) 40. The various grades of Type C - NRMB are (i) NRMB 120, (ii) NRMB 70 and (iii) NRMB 40. The grades of Type D - CRMB are (i) CRMB 50, (ii) CRMB 55 and (iii) CRMB 60. In case of CRMB, the numeral indicates the minimum desirable softening point value, viz., CRMB 50 means CRMB having minimum softening point value of 50 °C.

**Tests on modified bituminous binders**

**Elastic recovery test**

The elastic recovery test is intended to assess the degree of bitumen modification by elastomeric additives. This is a simple test conducted in a ductility testing machine to optimise the dosage of polymeric additives in bitumen and also helps in assessing
the quality of the modified bitumen in the laboratory. The elastic recovery of the modified bitumen is evaluated by comparing the recovery of a thread of modified bitumen after conditioning for one hour at specified test temperature.

The sample is prepared and conditioned as per the procedure of the ductility test but, in the elastic recovery mould shown in Fig. 6.25. The test specimen is elongated at the specified rate of 50 ± 2.5 mm per minute at the specified temperature to a deformation of 10 cm. As soon as the specimen is elongated to a deformation of 10 cm, the specimen is cut into two halves at the midpoint using scissors. The specimen is kept in the water bath in an undisturbed condition for a period of one hour at the specified temperature. After one hour, the elongated half of the specimen is moved back to the position near the fixed half of the test specimen, so that both the pieces of the specimen just touch each other. The length of the recombined specimen is measured as ‘D’ cm. The elastic recovery of the tested specimen in percent may be computed as:

\[
\text{Elastic recovery, } \% = \frac{100 (10 - D)}{10}
\]

(Eq. 6.11)

Here, D is the length of the combined specimen, cm

**Fig. 6.26 Elastic recovery test specimen and mould**

The elastic recovery test can be used to test the suitability of the modified bitumen for paving applications. Modified bitumen with low elastic recovery value is generally found to crack. As per the specifications of the Indian Roads Congress, the requirements of minimum elastic recovery value at 15°C for Elastomeric thermoplastic based polymer modified bitumen is minimum 75%; for plastomeric polymer modified bitumen is 50%; natural rubber modified bitumen is 30 - 50% and for crumb rubber modified bitumen the elastic recovery should be minimum 50%.

**Separation test**

In case of modified bitumen there is a possibility that the modifier used in the preparation of the modified bitumen may separate out during storage and transportation. The separation of the modifier such as polymer, crumb rubber, etc., during the hot storage condition is evaluated by comparing the ring and ball softening point values of the samples drawn from top and bottom of conditioned sealed tube of the modified bitumen. The conditioning consist of keeping a sealed tube of modified bitumen in vertical position at 163±5°C in an oven for a period of 48 hours.

The difference in softening points of the respective top and bottom samples is reported as the separation test value.
Effects of heat and air by thin film oven test

The bituminous binders harden when exposed to atmosphere. The test specimen is subjected to accelerated aging process by ‘Thin Film Oven Test’ (TFOT) under specified test conditions. The amount of hardening of the bituminous material is evaluated from the reduction in penetration test value, expressed as a percentage of the original penetration value.

Fraass breaking point test

Fraass breaking point is the temperature at which bitumen first becomes brittle as indicated by the appearance of cracks when a thin film of the bitumen on a metal plaque is cooled and flexed in accordance with the specified condition.

Complex modulus test

The complex modulus and phase angle define the resistance to deformation of the binder in the visco-elastic region. The complex modulus and phase angle are used to evaluate performance aspect of modified bitumen, where elastic recovery is insignificant. The test method may be summarised as: (a) preparation of test specimen (b) placement of the specimen in the rheometer maintained at the desired test temperature (c) selection of appropriate strain value and operation using the software and (d) reporting the recorded values of complex modulus (G*) and phase angle (sin δ).

Choice of modified bituminous binder

Modified bituminous binders are generally recommended for the roads with heavy traffic and the pavement subjected to over-loading conditions. The selection criteria for the type and grade of modified binder are based on atmospheric temperatures at the site of the project road. The softest recommended grade is PMB 120 or CRMB 50, which is used for cold climate areas. PMB 70 or CRMB 55 is used for moderate climate and PMB 40 or CRMB 60 is used for areas with hot climates. For cold climate areas, the properties such as penetration value at 4°C and Fraass breaking point value are to be taken into account.
# COMPARISON BETWEEN TAR & BITUMEN

<table>
<thead>
<tr>
<th>Bitumen</th>
<th>Tar</th>
</tr>
</thead>
<tbody>
<tr>
<td>It has black to dark brown color</td>
<td>It also has black to dark brown in color</td>
</tr>
<tr>
<td>It is natural petroleum product</td>
<td>Tar is produced by the destructive distillation of coal or wool</td>
</tr>
<tr>
<td>It is soluble in carbon disulphide &amp; in carbon tetrachloride</td>
<td>Tar is soluble only in toluene</td>
</tr>
<tr>
<td>It has better weather resisting property</td>
<td>It has inferior weather resisting property</td>
</tr>
<tr>
<td>Bitumen are less temp susceptible</td>
<td>Tar is more temp susceptible</td>
</tr>
<tr>
<td>Free carbon content is less</td>
<td>Free carbon content is More</td>
</tr>
<tr>
<td>It neither binds the aggregate well nor retains the presence of water</td>
<td>It binds aggregate more easily &amp; retain it better in the presence of water.</td>
</tr>
</tbody>
</table>
Introduction to pavement design

A highway pavement is a structure consisting of superimposed layers of processed materials above the natural soil sub-grade, whose primary function is to distribute the applied vehicle loads to the sub-grade. The pavement structure should be able to provide a surface of acceptable riding quality, adequate skid resistance, favorable light reflecting characteristics, and low noise pollution.

Requirements of a pavement

The pavement should meet the following requirements:

- Sufficient thickness to distribute the wheel load stresses to a safe value on the sub-grade soil
- Structurally strong to withstand all types of stresses imposed upon it
- Adequate coefficient of friction to prevent skidding of vehicles
- Smooth surface to provide comfort to road users even at high speed

Types of pavements

The pavements can be classified based on the structural performance into two, flexible pavements and rigid pavements. In flexible pavements, wheel loads are transferred by grain-to-grain contact of the aggregate through the granular structure. The flexible pavement, having less flexural strength, acts like a flexible sheet (e.g. bituminous road). On the contrary, in rigid pavements, wheel loads are transferred to sub-grade soil by flexural strength of the pavement and the pavement acts like a rigid plate (e.g. cement concrete roads).

Flexible pavements

Flexible pavements will transmit wheel load stresses to the lower layers by grain-to-grain transfer through the points of contact in the granular structure (see Figure 19:1). The
wheel load acting on the pavement will be distributed to a wider area, and the stress decreases with the depth. Taking advantage of this stress distribution characteristic

The lower layers will experience lesser magnitude of stress and less quality material can be used. Flexible pavements are constructed using bituminous materials. These can be either in the form of surface treatments (such as bituminous surface treatments generally found on low volume roads) or, asphalt concrete surface courses (generally used on high volume roads such as national highways).

Types of Flexible Pavements

The following types of construction have been used in flexible pavement: x Conventional layered flexible pavement,

x Full - depth asphalt pavement, and

x Contained rock asphalt mat (CRAM).

Conventional flexible pavements are layered systems with high quality expensive materials are placed in the top where stresses are high, and low quality cheap materials are placed in
lower layers.

Full-depth asphalt pavements are constructed by placing bituminous layers directly on the soil subgrade. This is more suitable when there is high traffic and local materials are not available.

Contained rock asphalt mats are constructed by placing dense/open graded aggregate layers in between two asphalt layers. Modified dense graded asphalt concrete is placed above the subgrade will significantly reduce the vertical compressive strain on soil subgrade and protect from surface water.

**Rigid pavements**

Rigid pavements have sufficient flexural strength to transmit the wheel load stresses to a wider area below. A typical cross section of the rigid pavement is shown in Figure below. Compared to flexible pavement, rigid pavements are placed either directly on the prepared subgrade or on a single layer of granular or stabilized material.

Since there is only one layer of material between the concrete and the sub-grade, this layer can be called as base or sub-base course. In rigid pavement, load is distributed by the slab action, and the pavement behaves like an elastic plate resting on a viscous medium. Rigid pavements are constructed by Portland cement concrete (PCC) and should be analyzed by plate theory instead of layer theory,
Types of Rigid Pavements

Rigid pavements can be classified into four types: x Jointed plain concrete pavement (JPCP), x Jointed reinforced concrete pavement (JRCP), x Continuous reinforced concrete pavement (CRCP), and x Pre-stressed concrete pavement (PCP).

**Jointed Plain Concrete Pavement** is plain cement concrete pavements constructed with closely spaced contraction joints. Dowel bars or aggregate interlocks are normally used for load transfer across joints. They normally has a joint spacing of 5 to 10m.

**Jointed Reinforced Concrete Pavement** Although reinforcements do not improve the structural capacity significantly, they can drastically increase the joint spacing to 10 to 30m. Dowel bars are required for load transfer. Reinforcements help to keep the slab together even after cracks. Continuous Reinforced Concrete Pavement Complete elimination of joints are achieved by reinforcement.

Factors affecting pavement design

Traffic and loading
Traffic is the most important factor in the pavement design. The key factors include contact pressure, wheel load, axle configuration, moving loads, load, and load repetitions.

**Contact pressure**

The tire pressure is an important factor, as it determines the contact area and the contact pressure between the wheel and the pavement surface. Even though the shape of the contact area is elliptical, for sake of simplicity in analysis, a circular area is often considered.

**Wheel load**

The next important factor is the wheel load which determines the depth of the pavement required to ensure that the subgrade soil is not failed. Wheel configuration affects the stress distribution and deflection within a pavement. Many commercial vehicles have dual rear wheels which ensure that the contact pressure is within the limits. The normal practice is to convert dual wheel into an equivalent single wheel load so that the analysis is made simpler.

**Axle configuration**

The load carrying capacity of the commercial vehicle is further enhanced by the introduction of multiple axles.

**Moving loads**

The damage to the pavement is much higher if the vehicle is moving at creep speed. Many studies show that when the speed is increased from 2 km/hr to 24 km/hr, the stresses and deflection reduced by 40 per cent.

**Repetition of Loads**

The influence of traffic on pavement not only depends on the magnitude of the wheel load, but also on the frequency of the load applications. Each load application causes some deformation and the total deformation is the summation of all these
Environmental factors

Environmental factors affect the performance of the pavement materials and cause various damages. Environmental factors that affect pavement are of two types, temperature and precipitation.

Equivalent single wheel load

To carry maximum load within the specified limit and to carry greater load, dual wheel, or dual tandem assembly is often used. Equivalent single wheel load (ESWL) is the single wheel load having the same contact pressure, which produces same value of maximum stress, deflection, tensile stress or contact pressure at the desired depth. The procedure of finding the ESWL for equal stress criteria is provided below. This is a semi-rational method, known as Boyd and Foster method, based on the following assumptions:

- equalancy concept is based on equal stress;
- contact area is circular;
- influence angle is 45°; and
- soil medium is elastic, homogeneous, and isotropic half space.

The ESWL is given by:

\[
\log_{10} ESWL = \log_{10} P + \frac{0.301 \log_{10} \left( \frac{2S}{2d+2z} \right)}{\log_{10} \left( \frac{2S}{2d+2z} \right)}
\]

Where P is the wheel load, S is the center to center distance between the two wheels, d is the clear distance between two wheels, and z is the desired depth.

Equivalent single axle load

Vehicles can have many axles which will distribute the load into different axles, and in turn to the pavement through the wheels. A standard truck has two axles, front axle with two wheels and rear axle with four wheels. But to carry large loads multiple axles are provided.
Since the design of flexible pavements is by layered theory, only the wheels on one side needed to be considered. On the other hand, the design of rigid pavement is by plate theory and hence the wheel load on both sides of axle need to be considered. Legal axle load:

**Repetition of axle loads:**

The deformation of pavement due to a single application of axle load may be small but due to repeated application of load there would be accumulation of unrecovered or permanent deformation which results in failure of pavement.

**Equivalent axle load factor:**

An equivalent axle load factor (EALF) defines the damage per pass to a pavement by the ith type of axle relative to the damage per pass of a standard axle load. While finding the EALF, the failure criterion is important. Two types of failure criteria’s are commonly adopted: fatigue cracking and rutting. The fatigue cracking model has the following form:

\[ N_f = f_1 (e_t)^{-f_2} \times (E)^{-f_3} \text{ or } N_f \propto e_t^{-f_2} \]

Where, \( N_f \) is the number of load repetition for a certain percentage of cracking, \( e_t \) is the tensile strain at the bottom of the binder course, \( E \) is the modulus of elasticity, and \( f_1, f_2, f_3 \) are constants. If we consider fatigue
cracking as failure criteria, and a typical value of 4 for $f_2$, then:

$$EALF = \left( \frac{e_i}{e_{\text{std}}} \right)^4$$

Where, $i$ indicate $i^{th}$ vehicle, and std indicate the standard axle. Now if we assume that the strain is proportional to the wheel load,

$$EALF = \left( \frac{W_i}{W_{\text{std}}} \right)^4$$

Similar results can be obtained if rutting model is used, which is:

$$N_d = f_4 (e_c)^{-f_5}$$

where $N_d$ is the permissible design rut depth (say 20mm), $s$ the compressive strain at the top of the subgrade, and $f_4; f_5$ are constants. Once we have the EALF, then we can get the ESAL as given below.

Equivalent single axle load, $\text{ESAL} =$

$$\text{Equivalent single axle load, ESAL} = \sum_{i=1}^{m} F_i n_i$$

Where, $m$ is the number of axle load groups, $F_i$ is the EALF for $i^{th}$ axle load group, and $n_i$ is the number of passes of $i^{th}$ axle load group during the design period.

Example Let number of load repetition expected by 80 KN standard axle is 1000, 160 KN is 100 and 40 KN is 10000. Find the equivalent axle load. Solution:
IRC method of design of flexible pavements

Design traffic

The method considers traffic in terms of the cumulative number of standard axles (8160 kg) to be carried by the pavement during the design life. This requires the following information:

1. Initial traffic in terms of CVPD
2. Traffic growth rate during the design life
3. Design life in number of years
4. Vehicle damage factor (VDF)
5. Distribution of commercial traffic over the carriage way.

Initial traffic

Initial traffic is determined in terms of commercial vehicles per day (CVPD). For the structural design of the pavement only commercial vehicles are considered assuming laden weight of three tones or more and their axle loading will be considered. Estimate of the initial daily average traffic flow for any road should normally be based on 7-day 24-hour classified traffic counts (ADT). In case of new roads, traffic estimates can be made on the basis of potential land use and traffic on existing routes in the area.
Traffic growth rate

Traffic growth rates can be estimated by studying the past trends of traffic growth, and By establishing econometric models. If adequate data is not available, it is recommended that an average annual growth rate of 7.5 percent may be adopted.

Design life

For the purpose of the pavement design, the design life is defined in terms of the cumulative number of standard axles that can be carried before strengthening of the pavement is necessary. It is recommended that pavements for arterial roads like NH, SH should be designed for a life of 15 years, EH and urban roads for 20 years and other categories of roads for 10 to 15 years.

Vehicle Damage Factor

The vehicle damage factor (VDF) is a multiplier for converting the number of commercial vehicles of different axle loads and axle configurations to the number of standard axle-load repetitions. It is defined as equivalent number of standard axles per commercial vehicle. The VDF varies with the axle configuration, axle loading, terrain, type of road, and from region to region. The axle load equivalency factors are used to convert different axle load repetitions into equivalent standard axle load repetitions. For these equivalency factors refer IRC: 37 2001. The exact VDF values are arrived after extensive field surveys.

Vehicle distribution

A realistic assessment of distribution of commercial traffic by direction and by lane is
necessary as it directly affects the total equivalent standard axle load application used in the
design. Until reliable data is available, the following distribution may be assumed.

x **Single lane roads:** Traffic tends to be more channelized on single roads than two lane roads and
to allow for this concentration of wheel load repetitions, the design should be based on total
number of commercial vehicles in both directions.

x **Two-lane single carriageway roads:** The design should be based on 75% of the commercial
vehicles in both directions.

x **Four-lane single carriageway roads:** The design should be based on 40% of the total
number of commercial vehicles in both directions.

x **Dual carriageway roads:** For the design of dual two-lane carriageway roads should be based
on 75% of the number of commercial vehicles in each direction. For dual three-lane
carriageway and dual four-lane carriageway the distribution factor will be 60% and 45%
respectively.

Numerical example
Design the pavement for construction of a new bypass with the following data:
1. Two lane carriageway
2. Initial traffic in the year of completion of construction = 400 CVPD (sum of both
directions)
3. Traffic growth rate = 7.5 %
4. Design life = 15 years
5. Vehicle damage factor based on axle load survey = 2.5 standard axle per commercial
vehicle
6. Design CBR of subgrade soil = 4%.
Solution

1. Distribution factor = 0.75

2. 
   \[
   N = \frac{365 \times [(1 + 0.075)^{15} - 1]}{0.075} \times 400 \times 0.75 \times 2.5
   \]
   
   \[
   = 7200000
   \]
   
   \[
   = 7.2 \text{ msa}
   \]

3. Total pavement thickness for CBR 4% and traffic 7.2 msa from IRC:37 2001 chart1 = 660 mm

4. Pavement composition can be obtained by interpolation from Pavement Design Catalogue (IRC:37 2001).
   
   (a) Bituminous surfacing = 25 mm SDBC + 70 mm DBM
   
   (b) Road-base = 250 mm WBM
   
   (c) sub-base = 315 mm granular material of CBR not less than 30 %

Rigid pavement design

Wheel load stresses - Westergaard's stress equation

The cement concrete slab is assumed to be homogeneous and to have uniform elastic properties with vertical sub-grade reaction being proportional to the deflection. Westergaard developed relationships for the stress at interior, edge and corner regions, denoted as \(_i\); \(_e\); \(_c\) in kg/cm\(^2\) respectively and given by the equation.

\[
\sigma_i = \frac{0.316 P}{h^2} \left[ 4 \log_{10} \left( \frac{l}{b} \right) + 1.069 \right]
\]

\[
\sigma_e = \frac{0.572 P}{h^2} \left[ 4 \log_{10} \left( \frac{l}{b} \right) + 0.359 \right]
\]

\[
\sigma_c = \frac{3 P}{h^2} \left[ 1 - \left( \frac{a \sqrt{2}}{l} \right)^{0.6} \right]
\]

where \(h\) is the slab thickness in cm, \(P\) is the wheel load in kg, \(a\) is the radius of the wheel load distribution in cm, \(l\) the radius of the relative stiffness in cm 29.1 and \(b\) is the radius of the resisting section in cm.
Temperature stresses

Temperature stresses are developed in cement concrete pavement due to variation in slab temperature. This is caused by (i) daily variation resulting in a temperature gradient across the thickness of the slab and (ii) seasonal variation resulting in overall change in the slab temperature. The former results in warping stresses and the latter in frictional stresses.

Warping stress

The warping stress at the interior, edge and corner regions, denoted as $\sigma_{ti}$, $\sigma_{te}$, $\sigma_{tc}$ in kg/cm² respectively and given by the equation

$$\sigma_{ti} = \frac{Et}{2} \left( \frac{C_x + \mu C_y}{1 - \mu^2} \right)$$

$$\sigma_{te} = \text{Max} \left( \frac{C_x Eyt}{2}, \frac{C_y Eyt}{2} \right)$$

$$\sigma_{tc} = \frac{Et}{3(1 - \mu)} \sqrt{\frac{a}{l}}$$

where $E$ is the modulus of elasticity of concrete in kg/cm² ($3 \times 10^5$), $\epsilon$ is the thermal coefficient of concrete per $^\circ$C ($1 \times 10^5$) $t$ is the temperature difference between the top and bottom of the slab, $C_x$ and $C_y$ are the coefficient based on $L_x/l$ in the desired direction and $L_y/l$ right angle to the desired direction, $\mu$ is the Poisson’s ration (0.15), $a$ is the radius of the contact area and $l$ is the radius of the relative stiffness.

Frictional stresses
The frictional stress $\sigma_f$ in kg/cm² is given by the equation.

$$\sigma_f = \frac{WLF}{2 \times 10^4}$$

Where $W$ is the unit weight of concrete in kg/cm² (2400), $f$ is the coefficient of sub grade friction (1.5) and $L$ is the length of the slab in meters.

**Combination of stresses**

The cumulative effect of the different stress give rise to the following three critical cases:

- **Summer, mid-day:** The critical stress is for edge region given by $\alpha_{\text{critical}} = \alpha_e + \alpha_{te} - \alpha_f$
- **Winter, mid-day:** The critical combination of stress is for the edge region given by $\alpha_{\text{critical}} = \alpha_e + \alpha_{te} + \alpha_f$
- **Mid-nights:** The critical combination of stress is for the corner region given by $\alpha_{\text{critical}} = \alpha_c + \alpha_{tc}$