Chapter 7

Design of Highway Pavements

7.1 INTRODUCTION

7.1.1 Objects and Requirements of Pavements

The surface of the roadway should be stable and non-yielding, to allow the heavy wheel loads of road traffic to move with least possible rolling resistance. The road surface should also be even along the longitudinal profile to enable the fast vehicles to move safely and comfortably at the design speed. The earth road may not be able to fulfil any of the above requirements, especially during the varying conditions of traffic loads and the weather. At high moisture contents, the soil becomes weaker and soft and starts yielding under heavy wheel loads, thus increasing the tractive resistance. The unevenness and undulations of the surface along the longitudinal profile of the road causes vertical oscillations in the fast moving automobiles, increasing the fuel consumption and the wear of the vehicle components, resulting in a considerable increase in the vehicle operation cost. Apart from this uneven pavement surface causes discomfort and fatigue to the passengers of fast moving vehicles and cyclists. Therefore, in order to provide a stable and even surface for the traffic, the roadway is provided with a suitably designed and constructed pavement structure. Thus a pavement consisting of a few layers of pavement materials is constructed over a prepared soil subgrade to serve as a carriageway.

The pavement carries the wheel loads and transfer the load stresses through a wider area on the soil subgrade below. Thus the stresses transferred to the subgrade soil through the pavement layers are considerably lower than the contact pressure or compressive stresses under the wheel load on the pavement surface. The reduction in the wheel load stress due to the pavement depends both on its thickness and the characteristics of the pavement layers. A pavement layer is considered more effective or superior, if it is able to distribute the wheel load stress through a larger area per unit depth of the layer. However, there will be a small amount of temporary deformation even on a good pavement surface when heavy wheel loads are applied. One of the objectives of a well designed and constructed pavement is therefore to keep this elastic deformation of the pavement within the permissible limits, so that the pavement can sustain a large number of repeated load applications during the design life.

Based on the vertical alignment and the environmental conditions of the site, the pavement may be constructed over an embankment, cut or almost at the ground level itself. It is always desirable to construct the pavement well above the maximum level of the ground water to keep the subgrade relatively dry even during monsoons.

7.1.2 Types of Pavement Structure

Based on the structural behaviour, pavements are generally classified into two categories :

(i) Flexible pavements

(ii) Rigid pavements

Flexible pavements

Flexible pavements are those, which on the whole have low or negligible flexural strength and are rather flexible in their structural action under the loads. The flexible pavement layers reflect the deformation of the lower layers on-to the surface of the layer. Thus if the lower layer of the pavement or soil subgrade is undulated, the flexible pavement surface also gets undulated. A typical flexible pavement consists of four components : (i) soil subgrade (ii) sub-base course (iii) base course and (iv) surface course. (See Fig. 7.1 a).

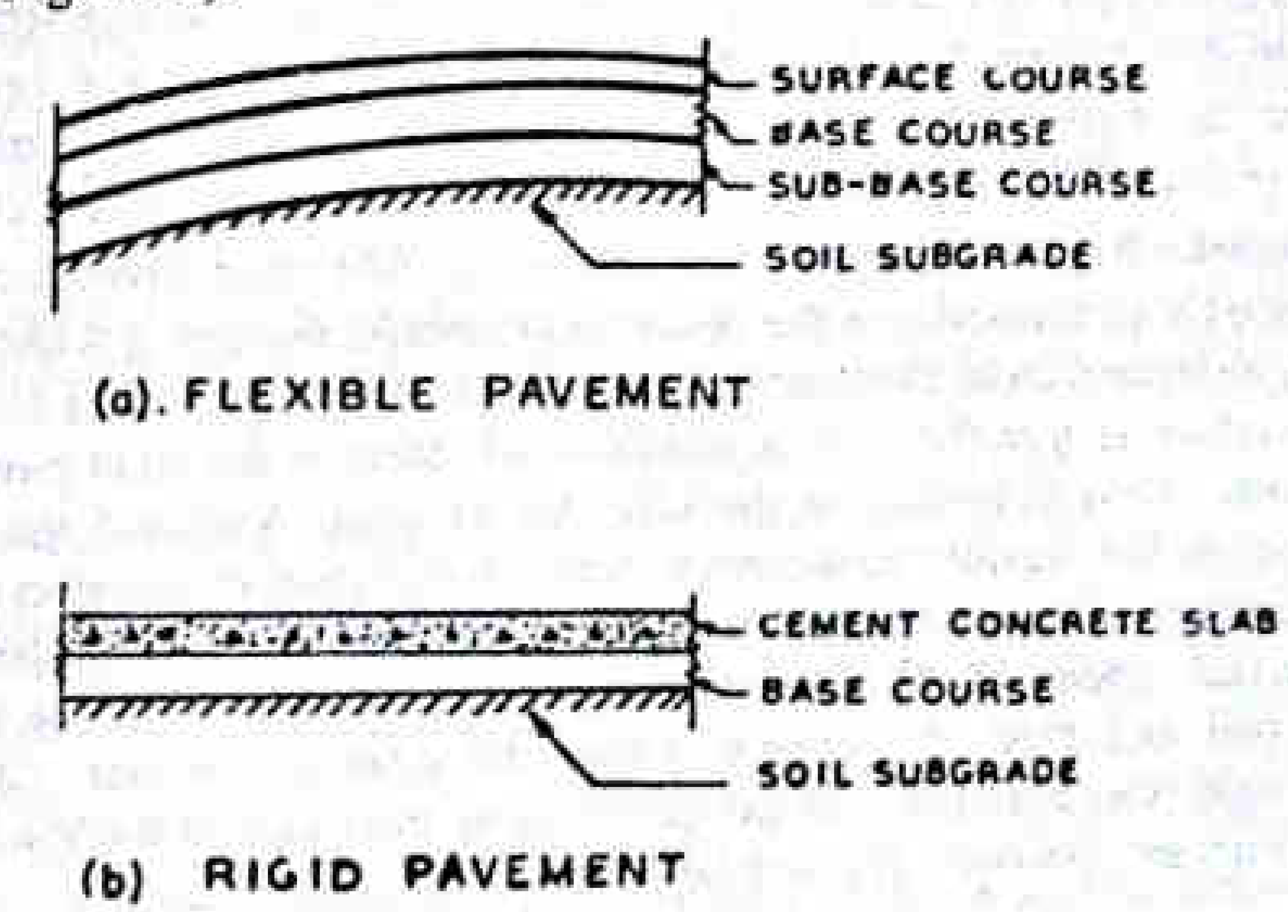


Fig. 7.1 Components of Flexible and Rigid Pavements

The flexible pavement layers transmit the vertical or compressive stresses to the lower layers by grain to grain transfer through the points of contact in the granular structure. A well compacted granular structure consisting of strong graded aggregate (interlocked aggregate structure with or without binder materials) can transfer the compressive stresses through a wider area and thus forms a good flexible pavement layer. The load spreading ability of this layer therefore depends on the type of the materials and the mix design factors. Bituminous concrete is one of the best flexible pavement layer materials. Other materials which fall under the group are, all granular materials with or without bituminous binder, granular base and sub-base course materials like the Water Bound Macadam, crushed aggregate, gravel, soil-aggregate mixes etc.

The vertical compressive stress is maximum on the pavement surface directly under the wheel load and is equal to the contact pressure under the wheel. Due to the ability to distribute the stresses to a larger area in the shape of a truncated cone, the stresses get decreased at the lower layers. Therefore by taking full advantage of the stress distribution characteristics of the flexible pavement, the *layer system concept* was developed. According to this, the flexible pavement may be constructed in a number of layer and the top layer has to be the strongest as the highest compressive stresses are to be sustained by this layer, in addition to the wear and tear due to the traffic. The lower layers have to take up only lesser magnitudes of stresses and there is no direct wearing action due to traffic loads, therefore inferior materials with lower cost can be used in the lower layers. The lowest layer is the prepared surface consisting of the local soil itself, called the subgrade. A typical cross section of a flexible pavement structure is shown in Fig. 7.1 (a); this consists of a wearing surface at the top, below which is the base course followed by the sub-base course and the lowest layer consists of the soil subgrade which has the lowest stability among the four typical flexible pavement components. Each of the flexible pavement layers above the subgrade, viz. sub-base, base course and the surface course may consist of one or more number of layers of the same or slightly different materials and specifications.

Flexible pavements are commonly designed using empirical design charts or equations taking into account some of the design factors. There are also semi-empirical and theoretical design methods.

Rigid pavements

Rigid pavements are those which possess note worthy flexural strength or flexural rigidity. The stresses are not transferred from grain to grain to the lower layers as in the case of flexible pavement layers. The rigid pavements are made of Portland cement concrete-either plain, reinforced or prestressed concrete. The plain cement concrete slabs are expected to take-up about 40 kg/cm^2 flexural stress. The rigid pavement has the slab action and is capable of transmitting the wheel load stresses through a wider area below. The main point of difference in the structural behaviour of rigid pavement as compared to the flexible pavement is that the critical condition of stress in the rigid pavement is the maximum flexural stress occurring in the slab due to wheel load and the temperature changes where-as in the flexible pavement it is the distribution of compressive stresses. As the rigid pavement slab has tensile strength, tensile stresses are developed due to the bending of the slab under wheel load and temperature variations. Thus the types of stresses developed and their distribution within the cement concrete slab are quite different. The rigid pavement does not get deformed to the shape of the lower surface as it can bridge the minor variations of lower layer.

The cement concrete pavement slab can very well serve as a wearing surface as well an effective base course. Therefore usually the rigid pavement structure consists of a cement concrete slab, below which a granular base or sub-base-course may be provided (see Fig. 7.1 b). Though the cement concrete slab can also be laid directly over the soil subgrade, this is not preferred particularly when the subgrade, consists of fine grained soil. Providing a good base or sub-base course layer under the cement concrete slab, increases the pavement life considerably and therefore works out more economical in the long run. The rigid pavements are usually designed and the stresses are analysed using the elastic theory, assuming the pavement as an elastic plate resting over an elastic or a viscous foundation.

Semi-rigid pavements

When bonded materials like the pozzolanic concrete (lime-flyash-aggregate mix), lean cement concrete or soil-cement are used in the base course or sub-base course layer the

pavement layer has considerably higher flexural strength than the common flexible pavement layers. However these bonded materials do not possess as much flexural strength as the cement concrete pavements. Therefore when this intermediate class of materials are used in the base or sub-base course layer of the pavements, they are called semi-rigid pavements. This third category of semi-rigid pavements are either designed as flexible pavements with some correction factors to find the thickness requirements based on experience, or by using a new design approach. These semi-rigid pavement materials have low resistance to impact and abrasion and therefore are usually provided with flexible pavement surface course.

7.1.3 Functions of Pavement Components

Soil Subgrade and its Evaluation

The soil subgrade is a layer of natural soil prepared to receive the layers of pavement materials placed over it. The loads on the pavement are ultimately received by the soil subgrade for dispersion to the earth mass. It is essential that at no time, the soil subgrade is *overstressed*. It means that the pressure transmitted on the top of the subgrade is within the allowable limit, not to cause excessive stress condition or to deform the same beyond the elastic limit. Therefore it is desirable that atleast top 50 cm layer of the subgrade soil is well compacted under controlled conditions of optimum moisture content and maximum dry density. It is necessary to evaluate the strength properties of the soil subgrade. This helps the designer to adopt the suitable values of the strength parameter for design purposes and in case this supporting layer does not come upto the expectations, the same is treated or stabilized to suit the requirements.

Many tests are known for measuring the strength properties of the subgrades. Mostly the test are empirical and are useful for their correlation in the design. Some of the tests have been standardised for the use. The common strength tests for the evaluation of soil subgrade are :

California bearing ratio test

California resistance value test

Triaxial compression test and

Plate bearing test.

These tests have been explained in detail in Chapter 6 and in the book *Highway Materials Testing* by the authors.

California Bearing Ratio (CBR) test is a penetration test, evolved for the empirical method of flexible pavement design. The CBR test is carried out either in the laboratory on prepared specimens or in the field by taking in-situ measurements. This test is also carried out to evaluate the strength of other flexible pavement component materials.

California resistance value is found by using Hveem stabilometer. This test is used in an empirical method of flexible pavement design based on soil strength.

Though *triaxial test* is considered as the most important soil strength test, still the test is not very commonly used in structural design of pavements. This is because only a few theoretical methods make use of this triaxial test results.

The *plate bearing test* is carried out using a relatively large diameter plate to evaluate the load supporting capacity of supporting power of the pavement layers. The plate bearing test is used for determining the elastic modulus of subgrade and other pavement layers. The results of the plate bearing tests are used in flexible pavement design method like *McLeod* method and the method based on layer system analysis by *Burmister*. Also the test is used for the determination of modulus of subgrade reaction in rigid pavement analysis by *Westergaard's* approach.

Sub-base and Base Courses and their Evaluation

These layers are made of broken stones, bound or unbound aggregate. Some times in sub-base course a layer of stabilized soil or selected granular soil is also used. In some places boulder stones or bricks are also used as a sub-base or soling course. However at the sub-base course, it is desirable to use smaller size graded aggregates or soil-aggregate mixes or soft aggregates instead of large boulder stone soling course of brick on edge soling course, as these have no proper interlocking and therefore have lesser resistance to sinking into the weak subgrade soil when wet. When the subgrade consists of fine grained soil and when the pavement carries heavy wheel loads, there is a tendency for these boulder stones or bricks to penetrate into the wet soil, resulting in the formation of undulations and uneven pavement surface in flexible pavements. Sub-base course primarily has the similar function as of the base course and is provided with inferior materials than of base course. The functions of the base course vary according to type of pavement.

Base course and sub-base courses are used under flexible pavement primarily to improve the load supporting capacity by distributing the load through a finite thickness. Base courses are used under rigid pavement for :

(i) preventing pumping

(ii) protecting the subgrade against frost action.

Thus the fundamental purpose of a base course and sub-base course is to provide a stress transmitting medium to spread the surface wheel loads in such manner as to prevent shear and consolidation deformations.

The sub-base and base course layers may be evaluated by suitable strength or stability test like plate bearing, CBR or stabilometer test. Each test has its own advantages and limitations. Some times these layers are evaluated in terms of pressure distribution characteristics.

Wearing Course and its Evaluation

The purpose of the wearing course is to give a smooth riding surface that is dense. It resists pressure exerted by tyres and takes up wear and tear due to the traffic. Wearing course also offers a water tight layer against the surface water infiltration. In flexible pavement, normally a bituminous surfacing is used as a wearing course. In rigid pavements, the cement concrete acts like a base course as well as wearing course. There are many types of surface treatment employed as wearing course. The type of surface depends upon the availability of materials, plants and equipment and upon the magnitude of surface loads.

There is no test for evaluating the structural stability of the wearing course. However the bituminous mixes used in the wearing courses are tested for their suitability otherwise. The term stability is as associated with such evaluation. Most popular test in use is Marshall stability test wherein the optimum content of bitumen binder is worked out based on the stability density, VMA and VFB of the given gradings of the aggregate mixtures. The test has been discussed in article 6.4.2. Plate bearing test and *Benkelman Beam* test are also sometimes made use of for evaluating the wearing course and the pavement as a whole.

7.2 DESIGN FACTORS

7.2.1 Factors to be considered in Design of Pavements

Pavement design consists of two parts :

(i) mix design of materials to be used in each pavement component layer

(ii) thickness design of the pavement and the component layers.

The details of bituminous mix design are given in Chapter 6 and the design of soil-aggregate mixes and stabilized soil mixes are given in Chapter 9 of this book. The design factors and methods for the structural design of flexible and rigid pavements are presented in this chapter.

The various factors to be considered for the design of pavements are given below :

(i) Design wheel load

(ii) Subgrade soil

(iii) Climatic factors

(iv) Pavement component materials

(v) Environmental factors

(vi) Special factors in the design of different types of pavements.

The thickness design of pavement primarily depends upon the design wheel load. Higher wheel load obviously need thicker pavement, provided other design factors are the same. While considering the design wheel load, the effects of total static load on each wheel, multiple wheel load assembly (if any, like the dual or the dual-tandem wheel loads), contact pressure, load repetition and the dynamic effects of transient loads are to be taken into account. As the speed increases, the rate of application of the stress is also increased resulting in a reduction in the pavement deformation under the load; but on uneven pavements, the impact increases with speed. Some of the important design factors associated with the traffic wheel loads have been explained in the subsequent article.

The properties of the soil subgrade are important in deciding the thickness requirement of pavements. A subgrade with lower stability requires thicker pavement to protect it from traffic loads. The variations in stability and volume of the subgrade soil with moisture changes are to be studied as these properties are dependent on the soil characteristics. The stress-strain behaviour of the soil under static and repeated loads have also significance. Apart from the design, the pavement performance to a great extent depends on the subgrade soil properties and the drainage. The importance and desirable properties of subgrade soil have been explained in Art. 6.1.

Among the climatic factors, rain fall affects the moisture conditions in the subgrade and the pavement layers. The daily and seasonal variations in temperature has significance in the design and performance of rigid pavements and bituminous pavements. Where freezing temperatures are prevalent during winter, the possibility of frost action in the subgrade and the damaging effects should be considered at the design stage itself.

The stress distribution characteristics of the pavement component layers depend on characteristics of the materials used. The fatigue behaviour of these materials and their durability under adverse conditions of weather should also be given due consideration.

The environmental factors such as height of embankment and its foundations details, depth of cutting, depth of subsurface water table, etc. affect the performance of the pavement. The choice of the bituminous binder and the performance of the bituminous pavements depends on the variations in pavement temperature with the seasons in the region. The warping stresses in rigid pavements depend on the daily variations in temperature in the region and in the maximum difference in temperature between the top and bottom of the pavement slab.

In the case of semi-rigid pavement materials, the formation of shrinkage cracks, pattern and the mode of propogation and the fatigue behaviour under such adverse conditions of hair cracks are to be studied before arriving at a rational method of design for the semi-rigid pavements.

7.2.2 Design Wheel Load

The various wheel load factors to be considered in pavement design are :

(i) Maximum wheel load

(ii) Contact pressure

(iii) Dual or multiple wheel loads and equivalent single wheel load

(iv) Repetition of loads

Maximum wheel load

The wheel load configurations are important to know the way in which the loads of a given vehicle are applied on the pavement surface. Typical wheel load configuration of a tractor trailer unit of a heavy duty vehicle is shown in Fig. 7.2.

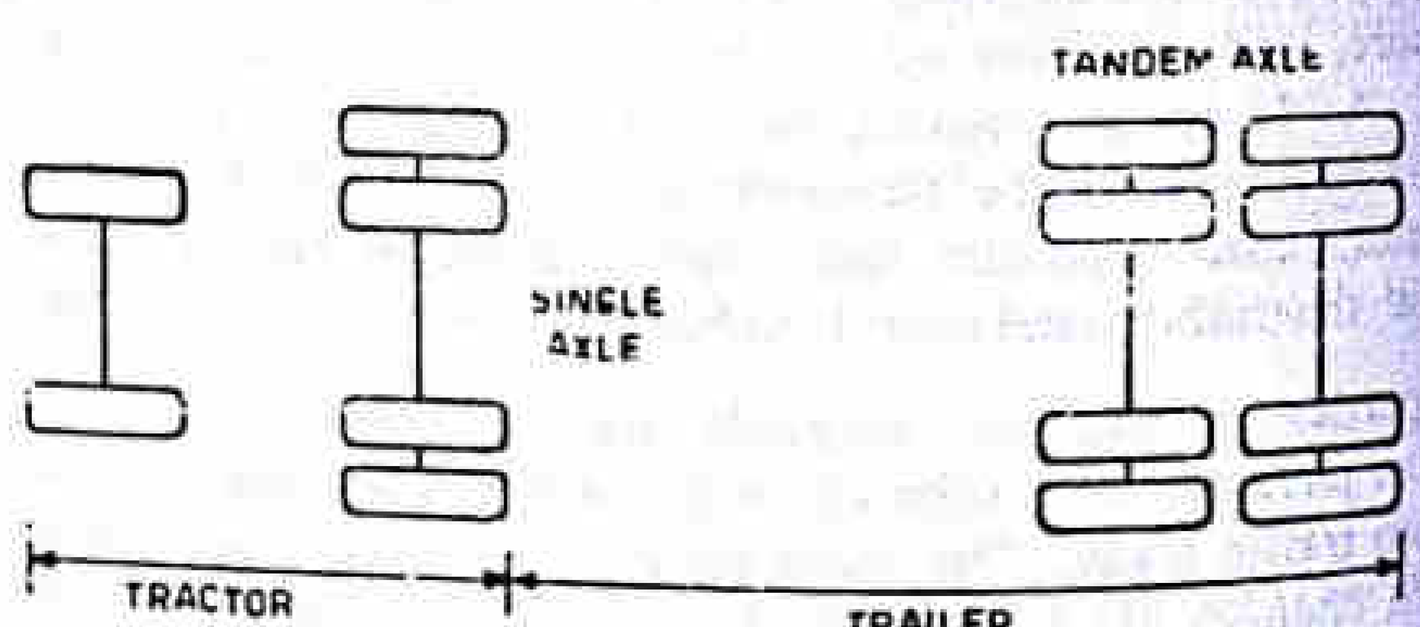


Fig. 7.2 Wheel Configuration of Tractor Trailer Unit

For highways the maximum legal axle load as specified by Indian Roads Congress is 8170 kg with a maximum equivalent single wheel load of 4085 kg. Total load influences the thickness requirements of pavements. Tyre pressure influences the quality of surface (wearing) course. In fact the magnitude of the vertical pressure at any depth of soil subgrade mass depends upon the surface pressure as well as on the total load.

The equation for vertical stress computations under a uniformly distributed circular load based on *Boussineq's* theory is given by :

$$\sigma_z = p\left[1 - \frac{z^3}{(a^2 + z^2)^{3/2}}\right] \tag{7.1}$$

Here σ_z = vertical stress at depth z

p = surface pressure

z = depth at which σ_z is computed

a = radius of loaded area

Using the above equation the variation of vertical stress with depth is plotted as given in Fig. 7.3.

Contact pressure

As seen from the Fig. 7.3 the influence of tyre pressure is predominating in the upper layers. At a greater depth the effect of tyre pressure diminishes and the total load exhibits a considerable influence on the vertical stress magnitudes. Tyre pressure of high magnitudes therefore demand high quality of materials in upper layers in pavements. The total depth of pavement is however not influenced by the tyre pressure. With constant tyre pressure, the total load governs the stress on the top of subgrade within allowable limits.

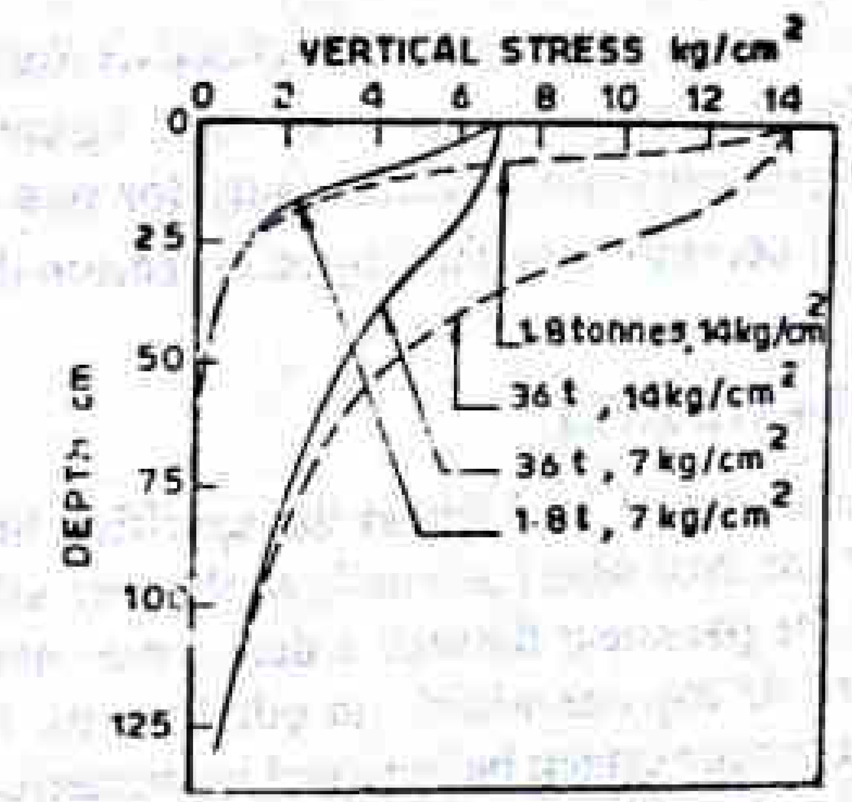


Fig. 7.3 Vertical Stress Distribution

The stresses on the pavement surface under the steel tyred wheels of bullock carts are very high. This demands use of very strong and hard aggregate for the wearing surface of the pavement. However the stresses at a lower layers of pavement due to the bullock cart wheel are negligibly small as the gross load is very small.

Generally the wheel load is assumed to be distributed over a circular area. But by measurement of the imprints of tyres with different load and inflation pressures, it is seen that contact areas in many cases are elliptical in shape. Three terms in use with reference to tyre pressure are :

Tyre pressure

Inflation pressure and

Contact pressure

Theoretically, all these terms should mean the same thing. Tyre pressure and inflation pressure mean exactly the same. The contact pressure is found to be more than tyre pressure when the tyre pressure is less than 7 kg/cm^2 and it is vice versa when the tyre pressure exceeds this value. Contact pressure can be measured by the relationship :

$$\text{Contact pressure} = \frac{\text{Load on wheel}}{\text{Contact area or area of imprint}}$$

The general variation between the tyre pressure and measured contact pressure is shown in Fig. 7.4.

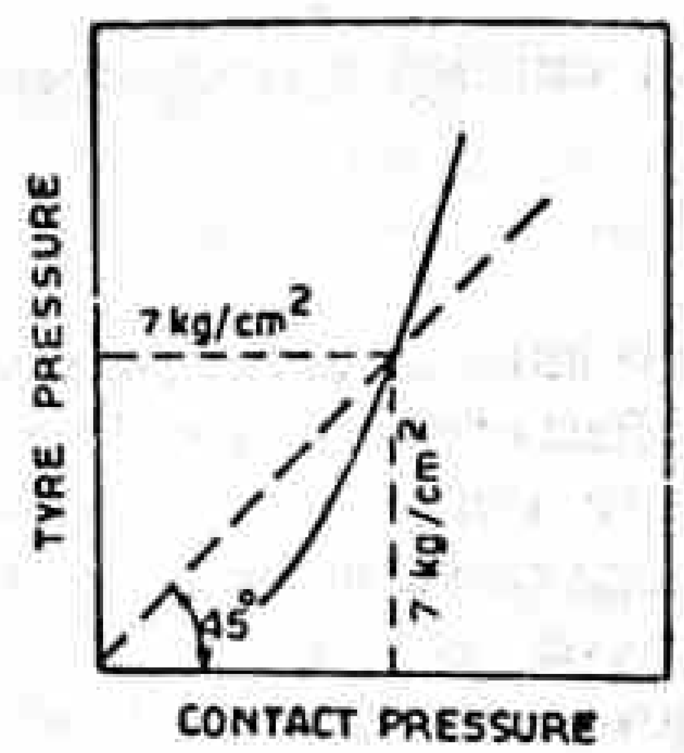


Fig. 7.4 Relationship between Tyre and Contact Pressure

The ratio of contact pressure to tyre pressure is defined as *Rigidity Factor*. Thus value of rigidity factor is 1.0 for an average tyre pressure of 7 kg/cm^2. This value is higher than unity for lower tyre pressures and less than unity for tyre pressures higher than 7 kg/cm^2. The rigidity factor depends upon the degree of tension developed in the walls of the tyres.

Equivalent single wheel load (ESWL)

To maintain the maximum wheel load within the specified limit and to carry greater load it is necessary to provide dual wheel assembly to the rear axles of the road vehicles. In doing so the effect on the pavement through a dual wheel assembly is obviously not equal to two times the load on any one wheel. In other words, the pressure at a certain depth below the pavement surface cannot be obtained by numerically adding the pressure caused by one wheel load. The effect is in between the single load and two times load carried by any one wheel. See Fig. 7.5. In order to simplify the analysis. The load dispersion is assumed to be at an angle of 45°. In the dual wheel load assembly, let d be the clear gap between the two wheels, S be the spacing between the centers of the wheels and a be the radius of the circular contact area of each wheel. Then S = (d + 2a).

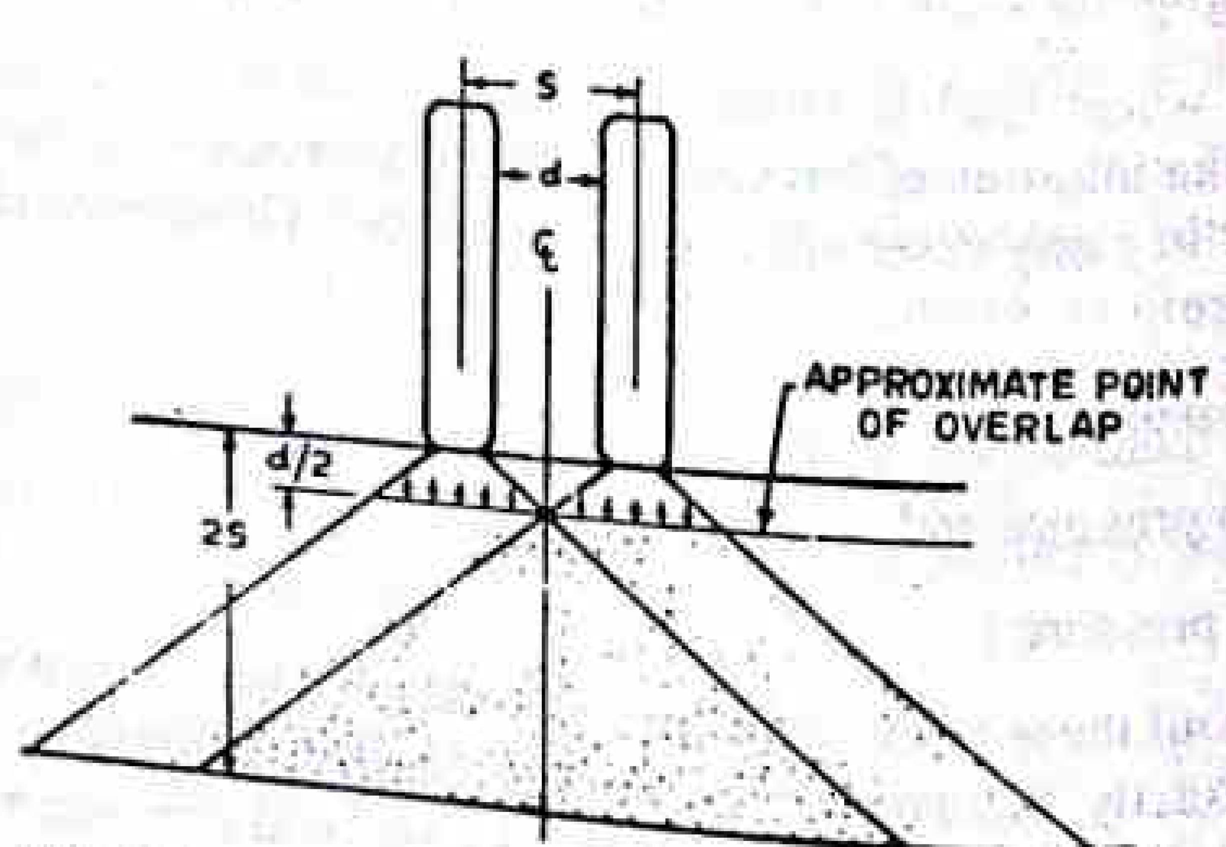


Fig. 7.5 Stress Overlap due to Dual Wheels

Upto the depth of d/2 each wheel load P acts independently and after this point the stresses induced due to each load begins to overlap. At depth 2S and above, the stresses induced are due to the effect of both wheels as the area of overlap is considerable. So the total stresses due to the dual wheels at any depth greater than 2S is considered to be equivalent to a single wheel load of magnitude 2P, though this stress is likely to be slightly greater than the stress due to the dual wheels.

Equivalent Single Wheel Load (ESWL) may be determined based on either equivalent deflection or equivalent stress criterion. Multiple wheel loads are converted to ESWL and this value is used in pavement design.

Suppose a dual wheel load assembly causes a certain value of maximum deflection Δ at a particular depth Z (say, depth equal to the thickness of the pavement). As per deflection criterion the ESWL is that single wheel load having the same contact pressure which produces the same value of maximum deflection at the depth Z. Similarly by stress criterion, the ESWL is the single wheel load producing the same value of maximum stress at the desired depth Z as the dual. The ESWL is usually determined by the equivalent stress criterion using a simple graphical method.

A straight line relationship is assumed between ESWL and depth on log-log scales. For determining ESWL the plot is made as shown in Fig. 7.6.

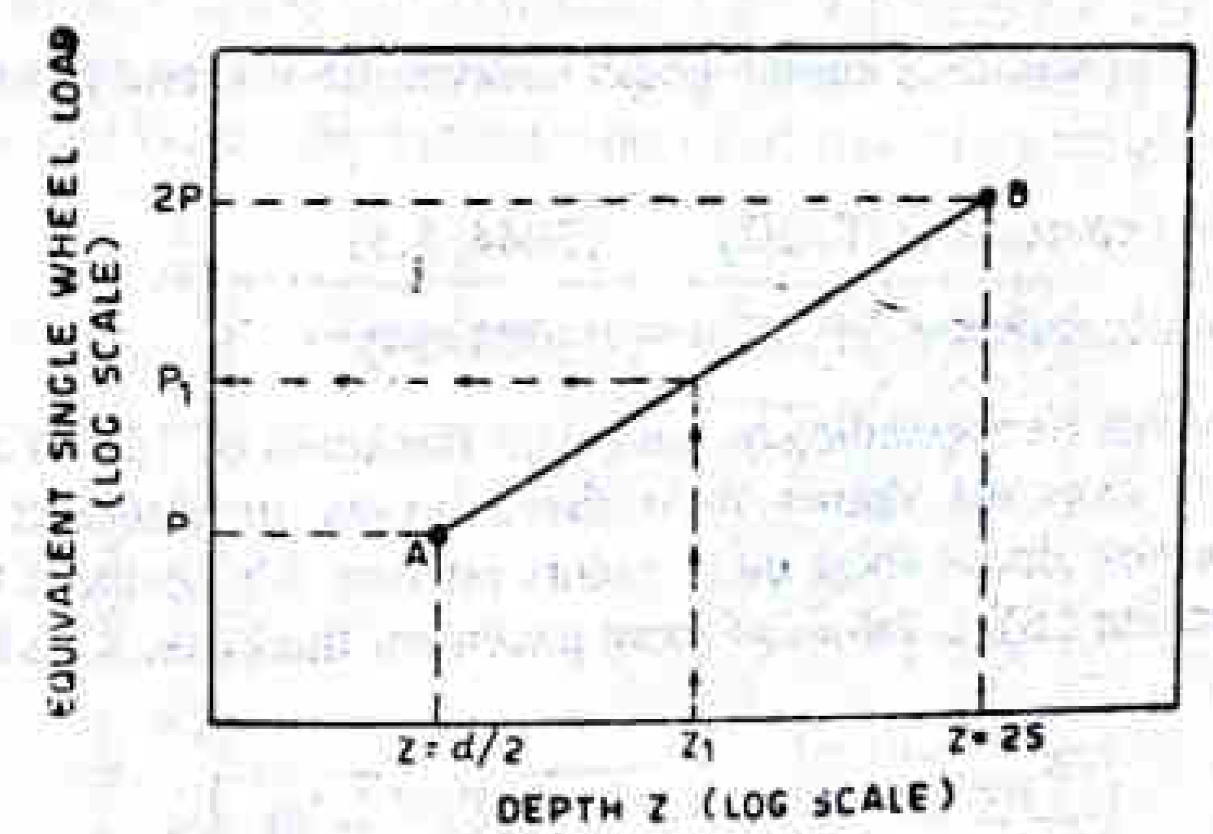


Fig. 7.6 Graphical Method for ESWL

Two points A and B are plotted on the log-log graph with coordinates of A (P, d/2) and B (2P, 2S). Line AB is a plot which is the locus of points where any single wheel load is equivalent to a certain set of dual wheels. To calculate the ESWL for a dual assembly, it is essential to estimate a design thickness of the pavement. Thus ESWL is obtained at the assumed thickness from this graph. The same is used in design calculations. If the design thickness so obtained is equal to the estimated thickness then the ESWL calculations could be considered as correct. Otherwise trials are made.

In heavy trucks and trailers, the load on each wheel may be further reduced by multiple wheels and tandem axles. Figure 7.2 shows an arrangement of dual wheels and tendem axles. It is possible to determine ESWL for such loading arrangements also.

Example 7.1

Calculate ESWL of a dual wheel assembly carrying 2004 kg each for pavement thickness of 15, 20 and 25 cms. Centre to centre tyre spacing = 27 cm and distance between the walls of the tyres = 11 cm.

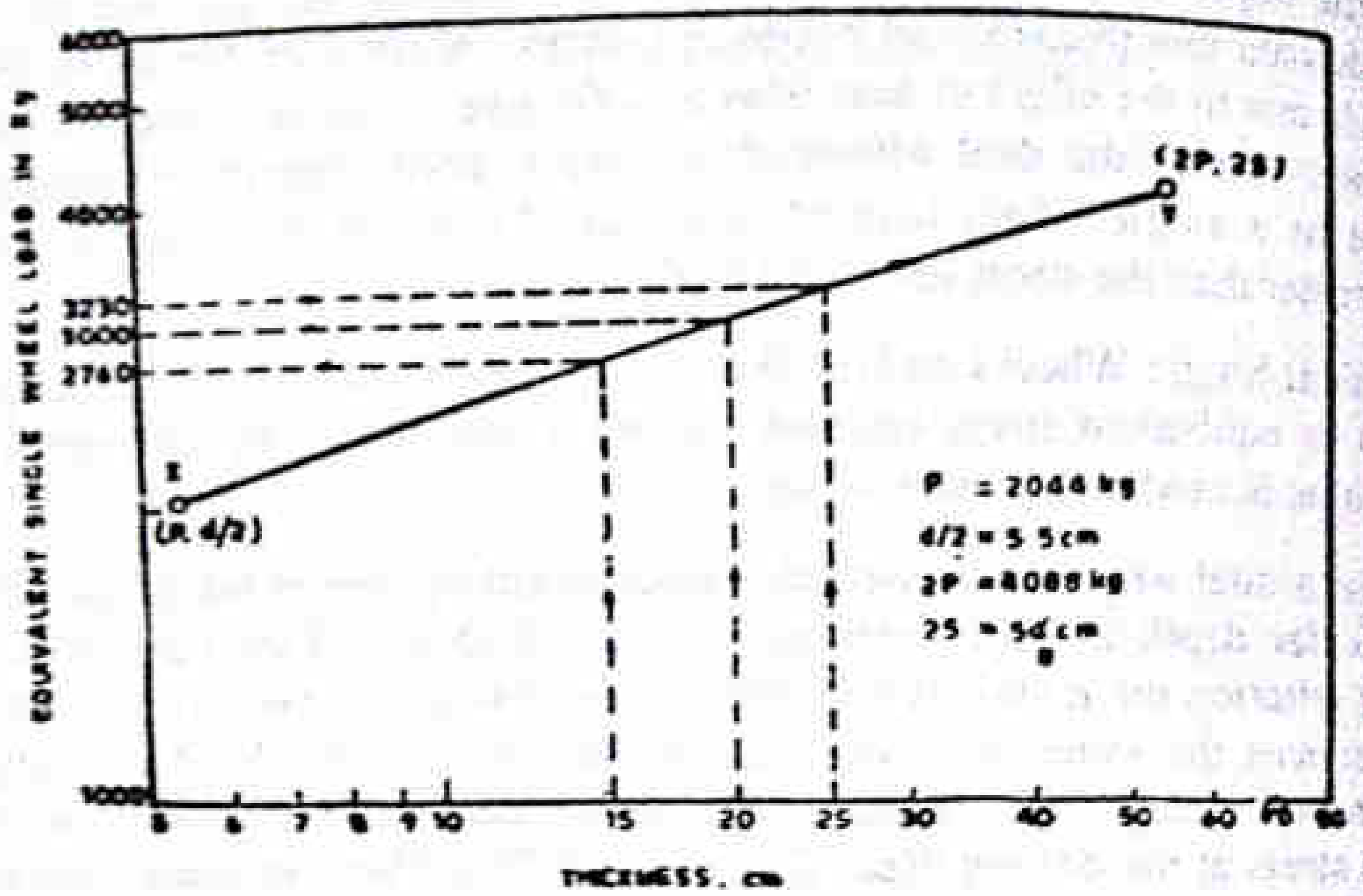


Fig. 7.7 ESWL Calculation (Example 7.1)

Solution

Here P = 2044 kg; 2P = 4088 kg; d = 11 cm; S = 27 cm

X and Y points are plotted on a log-log graph between ESWL and pavement thickness (See Fig. 7.7).

X has coordinates (P, d/2) = (2044, 5.5)

Y has coordinates (2P, 2S) = (4088, 54)

On the X-axis, points corresponding to pavement thickness of 15, 20 and 25 cms are marked and vertical lines are drawn from these points to intersect the line XY. Horizontal lines are now drawn from these points on line XY to meet the Y-axis, to obtain the corresponding ESWL values at these pavement thickness. ESWL values thus obtained are

Pavement thickness cm	ESWL kg
15	2760
20	3000
25	3230

Repetition of loads

The deformation of pavement or subgrade due to a single application of wheel load may be small. But due to repeated application of the load there would be increased magnitude of plastic and elastic deformations and the accumulated unrecovered or permanent deformations may even result in pavement failure.

It required to carry out traffic surveys for accounting the factor of repetitions for wheel loads in the design of pavement. Such data collected are converted to some constant equivalent wheel loads. Traffic composition in India is of mixed type and it is essential for design purposes to convert the various wheel loads to one single standard wheel load. Equivalent wheel load is a single load equivalent to the repeated applications of any particular wheel load on a pavement which requires the same thickness and strength of pavements.

If the pavement structure fails with N_1 number of repetitions of P_1 kg load and similarly if N_2 number of repetitions of P_2 kg load can also cause failure of the same pavement structure, then P_1N_1 and P_2N_2 are considered equivalent. *McLeod* has given a procedure for evolving *equivalent load factors* for designing flexible pavements.

McLeod assumes that the pavement thickness which are designed for a given wheel load would support one million repetition of such load during the life of pavement. For one load application, the pavement thickness so required is only one fourth the pavement thickness designed for 10^6 load repetitions.

For computing equivalent load factors, the plot similar to the one given in Fig. 7.8 was considered by McLeod. One fourth the design thickness were plotted for various wheel loads on vertical axis against one load application and total thickness (100%) were plotted on vertical axis drawn at 10^6 repetitions. The respective repetitions are then read from the figure for different loads at a pavement thickness of 25 cm (which is an average thickness for highway pavement on an ordinary soil subgrade). The values so obtained are given in Table 7.1. If the wheel load of 2268 kg (5000 lb) and the failure number of repetitions for 25 cm thick pavement are taken as standard, the number of failure repetitions for higher wheel loads may be obtained from Fig. 7.8. The number of failure repetitions for 2268 and 2722 kg are respectively 105,000 and 50,000 and so 2722 kg may be considered equivalent to 105,000/50,000 = 2.1 times the load value of 2268 kg. Hence the equivalent wheel load factor in this case is taken as 2.1 or say 2. The equivalent wheel load factors for various wheel load repetitions are given in Table 7.1.

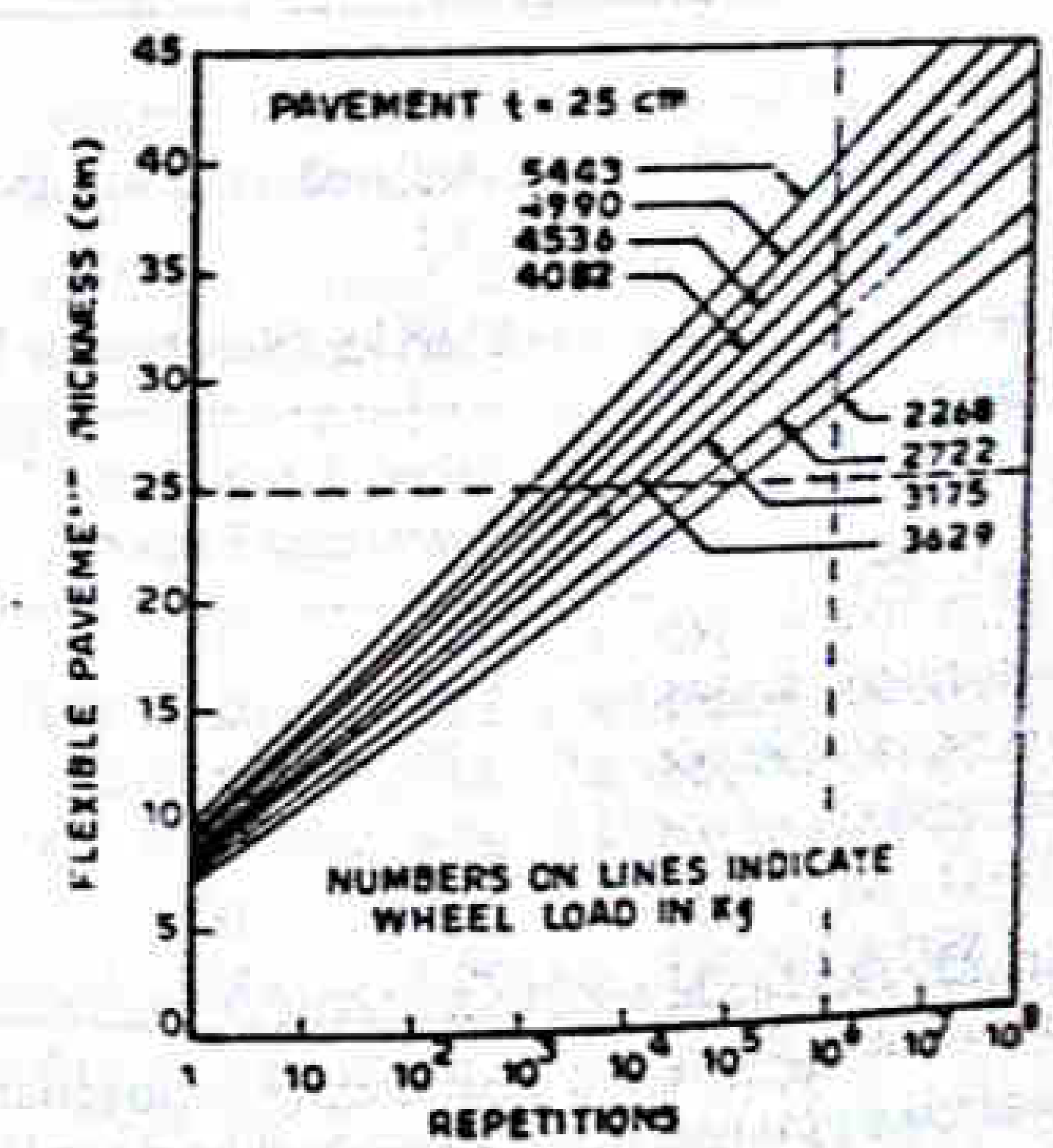


Fig. 7.8 Repetitions and Equivalent Load Factors

Equivalent load factors are employed to convert daily traffic count for each category of wheel load for design purposes.

Wyoming state Highway Department of USA employs the above concept for evaluating the design repetitions of wheel loads.

Table 7.1 Equivalent Wheel Load Factors

Wheel load kg	Repetitions to failure, number	Equivalent to 2268 kg	Equivalent load factors
2268	105,000	1.0	1
2722	50,000	2.0	2
3175	22,500	4.7	4
3629	13,000	8.2	8
4082	6,500	16.3	16
4536	3,300	32.0	32
4990	1,700	62.0	64
5443	1,000	105.0	128

Example 7.2

Calculate design repetitions for 20 years period for various wheel loads equivalent to 2268 kg wheel load using the following traffic survey data on a four lane road.

Wheel loads kg	Average Daily Traffic (both directions)	Percentage of total traffic volume
2268	Total volume (consideration traffic growth) 215	13.17
2722		15.30
3175		11.76
3629		14.11
4082		6.21
4536		5.84

Solution

Design repetitions for a period of 20 years calculated as given in Table 7.2. The equivalent load factors have been taken from Table 7.1

Table 7.2 Design Repetitions Equivalent to 2268 kg (Solution to Example 7.2)

Wheel loads kg	A.D.T. (both direction)	Percentage for each load	Days/ years	Number of years	Equivalent load Factors	Design repetitions equivalent of 2268 kg load
2268	215 ×	13.17/100 ×	365 ×	20	1	= 206,703
2722	215 ×	15.30/100 ×	365 ×	20	2	= 480,267
3175	215 ×	11.76/100 ×	365 ×	20	4	= 738,293
3629	215 ×	14.11/100 ×	365 ×	20	8	= 1,771,652
4082	215 ×	6.21/100 ×	365 ×	20	16	= 1,599,455
4536	215 ×	5.84/100 ×	365 ×	20	32	= 2,933,082
Total estimated repetitions (two directions)						= 7729,452
Design repetitions equivalents of 2268 kg wheel load per lane						= 7729,452/4 = 19,32,363

7.2.3 Strength Characteristics of Pavement Materials

For design purposes, it is required that the various pavement materials are assigned strength parameters suitable to the design method employed for the purpose. Various materials used in sub-base course and base course are evaluated by different tests as indicated in article 6.1.8. The general strength values evaluated are :

(i) California Bearing Ratio (CBR) value

(ii) Elastic moduli

California Bearing ratio

The test has been explained in article 6.1.8. The strength values so obtained for the materials tested are of relative significance and do not provide as absolute measure. There are design methods which employ the CBR strength values of materials used in different pavement layers.

Elastic Modulii

Depending upon the design methods, the elastic modulii of different pavement materials are evaluated. Mainly, plate bearing test is employed for this purpose. This has been explained in article 6.1.8. Further modulus elasticity or modulus of deformation of highway materials may be determined from triaxial compression test.

The elastic modulii values of the following are determined by plate bearing tests :

(i) Subgrade modulus

(ii) Elastic moduli of base course and sub-base course materials.

Subgrade modulus is computed from the plate bearing test data. Boussinesq's settlement equation for maximum vertical deflection Δ at the surface and the centre of a flexible plate is given by :

$$\Delta = \frac{1.5\,p\,a}{E_s} \qquad (7.2)$$

Here p is the uniform pressure on the flexible loaded plate of radius a. E_s is the modulus of elasticity of the soil.

If the load is applied by means of a rigid circular plate instead of flexible one, the pressure on the surface is not uniformly distributed and so the theoretical value of maximum deflection Δ at the surface in this case is given by :

$$\Delta = \frac{1.18\,p\,a}{E_s} \qquad (7.3)$$

Plate bearing test conducted with a mild steel plate is considered relevant to the condition of *rigid plate* as in Eq. 7.3. But the wheel loads through inflate rubber tyres may be considered as *flexible plate* loading or loading with uniformly distributed pressure.

If the level of design deflection is defined, then from the plate bearing test carried out on a given soil subgrade with the plate of diameter = 2a, the pressure p can be recorded from the test plots. From Eq. 7.3,

$$\text{Subgrade modulus } E_s = 1.18\,\frac{p\,a}{\Delta}$$

Further extending the definition of subgrade modulus, *Westergaarde* employed the strength parameter of soil subgrade in rigid pavement analysis considering it as *modulus of subgrade reaction* K. The computation of this value from plate bearing test values is explained in article 6.1.8.

For computing elastic modulii of pavement materials, *Burmister's elastic layered system analysis* is employed. The displacement equations given by Burmister for a two layer system consisting of a pavement layer of thickness h with elastic modulus E_p laid over the subgrade is given by :

$$\Delta = 1.5 \frac{p\,a}{E_s} . F_2 \text{ (For flexible plate)} \tag{7.4}$$

$$\Delta = 1.18 \frac{p\,a}{E_s} . F_2 \text{ (For rigid plate)} \tag{7.5}$$

With known values of design deflections, yielded pressure p, subgrade modulus E_s and radius of loaded area a, the value of displacement factor F_2 is obtained. F_2 is a dimensionless factor and depends on the ratio of modulii of elasticity of subgrade to pavement E_s/E_p as well as the depth of radius ratio, z/a.

Thus using relationship between F_2 and the ratio of pavement thickness to radius of contact area, h/a the modulii ratio of subgrade and pavement material, E_s/E_p is calculated (See Fig. 7.22 given later). Since the value of E_s is known, the value of elastic modulus of the pavement Ep is thus obtained.

7.2.4 Climatic Variations

The climatic variations cause following major effects.

(i) Variation in moisture condition

(ii) Frost action

(iii) Variation in temperature

The pavement performance is very much affected by the variation in moisture and the frost. This is mainly because of the variation in stability and the volume of the subgrade soil due to these two effects. Variation in temperature generally affects the pavement materials like bituminous mixes and cement concrete.

Variation in Moisture Content

Considerable variations in moisture condition of subgrade soil are likely during the year, depending on climatic conditions, soil type ground water level and its variations, drainage conditions, type of pavement and shoulders. The surface water during rains may enter the subgrade either through the pavement edges or through the pavement itself if it is porous. The subgrade moisture variations depend on fluctuations of ground water table. The moisture movement in subgrade is also caused by capillary action and vapour movement. However, high moisture variations could be controlled by providing suitable surface and sub-surface drainage system.

The stability of most of the subgrade soils are decreased under adverse moisture conditions. Presence of soil fraction with high plasticity will result in variations in volume (swelling and shrinkage) with variation in water content. As the moisture content of subgrade below the centre is often different from that at the pavement edges, there can be differential rise or fall of the pavement edges with respect to the centre, due to swelling and shrinkage of the subgrade soil. These effects are likely to cause considerable damages to the pavements and will also be progressive and cumulative.

Frost Action

Frost action refers to the adverse effective due to frost heave, frost melting or thaw and the alternate cycles of freezing and thawing. The frost action in general includes all effects associated with freezing temperature on pavement performance.

The held water in subgrade soil forms ice crystals at some spots if the freezing temperatures continue for a certain period. These ice crystals grow further in size if there is a continuous supply of water due to capillary action and the depressed temperature continues. This results in raising of portion of the pavement structure known as *frost heave*. If the frost heave cases uniform raising of pavement structure, the subgrade support is not adversely affected at this stage. However non-uniform heaving may cause damages.

Subsequent increase in temperature would result in melting or thawing of the frozen ice crystals and soften the road bed. The load carrying capacity of the subgrade is considerably decreased at this stage due to the voids created by the melted ice crystals and the excessive water trapped in the thawed soil below the pavement. Under heavy traffic, the pavement would deflect excessively causing progressive failure due to decreased load carrying capacity of the subgrade.

The freezing and thawing which occur alternately due to the variation in weather causes undulations and considerable damages to the pavement. Hence the overall effects due to frost heave, frost melting and alternate freeze-thaw cycles is called the *frost* action.

The various factors on which frost action depends may be broadly classified as :

(i) Frost susceptible soil

(ii) Depressed temperature below freezing point

(iii) Supply of water

(iv) Cover

The soil type, grain size distribution, permeability and capillarity of soil influence frost action. The temperature below freezing point and duration of the freezing temperature determines the depth up to which frost action exceeds. Unless there is a continuous supply of water, the small ice crystals formed can not grow in size. The supply of water may be from the ground water due to the capillary action or soil section. The rate of heat transfer depends on soil density and texture, moisture content and the proportion of frozen moisture in the soil mass under consideration. The type and colour of the cover affects the heat transfer from the atmosphere to the soil beneath the cover. For example temperature under a black top pavement will be higher than that under alight coloured pavement or base course.

One of the most effective and practical methods to decrease the damaging effects due to water and frost action is to install proper surface and subsurface drainage system. Construction of base, sub-base and top layer of subgrade, upto the desired depth, by granular and non-frost susceptible material with good drainage characteristics would go a long way in withstanding the adverse climatic conditions. Yet another effective method is by providing a suitable *capillary cut-off*. It is also possible to reduce the adverse effects of frost action on pavements by soil stabilization. The stabilized soil mix may be designed to withstand the adverse climatic conditions of alternate wet-dry and freeze-thaw cycles. Suitable stabilized soil mixes may be designed and provided for base course, sub-base courses and even at the top layer of subgrade. Salts like calcium chloride or sodium chloride when mixed with subgrade soil lowers the freezing temperature of the soil-water and hence temporarily decreases the intensity of frost action.

Variation in Temperature

Wide variation in temperature due to climatic changes may cause damaging effects in some pavements. Temperature stresses of high magnitude are induced in cement concrete pavements due to daily variations in temperature and consequent warping of the pavement as discussed in Article 7.4.3. Bituminous pavement become soft in hot weather and brittle in very cold weather.

From the above discussion, it is evident that the design and performance of pavements depend on the traffic loads, the subgrade, soil pavement materials and climatic conditions.

7.3 DESIGN OF FLEXIBLE PAVEMENTS

7.3.1 Flexible Pavement Design Methods

As discussed earlier, the flexible pavements are built with number of layers. In the design process, it is to be ensured that under the application of load none of the layers is overstressed. This means that at any instance no section of the pavement structure is subjected to excessive deformation to form a localized depression or settlement.

The maximum intensity of stresses occurs in the top layer of the pavement. The magnitude of load stresses reduces at lower layers. Hence the superior pavement materials are used in top layers of flexible pavements.

In the design of flexible pavements, it has yet not been possible to have a rational design method wherein design process and service behaviour of the pavement can be expressed or predicted theoretically by mathematical laws. Flexible pavement design methods are accordingly either empirical or semi-empirical. In these methods, the knowledge and experience gained on the behaviour of the pavements in the past are usefully utilized. The design methods therefore include methods based on soil classification like group index value and methods based on soil strength like California Bearing Ratio; California Resistance Value (R-value) and subgrade support based on plate bearing tests.

Besides these, the method based on stress-deformation characteristics of the pavement layers utilizing the theoretical considerations of elastic layered system analysis advocated by Burmister has considerable scope. An understanding of theoretical analysis by the designer is imperative since this helps to recognize the complexity of the phenomenon involved.

Various approaches of flexible pavement design may be thus classified into three broad groups.

(a) Empirical methods

(b) Semi-empirical or semi theoretical methods

(c) Theoretical methods

Empirical methods are either based on physical properties or strength parameters of soil subgrade. When the design is based on stress-strain function and modified based on experience, it may be called semi-empirical or semi-theoretical. There are design methods based on theoretical analysis and mathematical computations. Each one of the approaches has its own advantages and limitations.

Out of the various flexible pavement design methods available, the following are discussed here.

(i) Group Index method

(ii) California Bearing Ratio method

(iii) California R value or stabilometer method

(iv) Triaxial test method

(v) McLeod method

(vi) Burmister method

Of the design methods, the group Index, CBR, Stabilometer and McLeod methods are empirical methods. . The Triaxial test method is a theoretical method using empirical modifications as suggested by Kansas State Highway Department and therefore may be considered as a semi-empirical method. Burmister method is a theoretical approach using elastic two-layer theory.

7.3.2 Group Index Method

D. J. Steel in 1945 provided a discussion on the paper dealing with the Highway Research Board method of soil classification which included the suggested thickness requirements based on Group Index values. As discussed in article 6.1.6 the group index (GI) value is an arbitrary index assigned to the soil types in numerical equations based on the percent fines, liquid limit and plasticity index. Refer Eq. 6.1. The GI values of soils vary in the range of 0 to 20. The higher the GI value, weaker is the soil subgrade and for a constant value of traffic volume, the greater would be the thickness requirement of the pavement.

The design chart for Group Index method for determining the pavement thickness is given in Fig. 7.9. The traffic volume in this method is divided in three groups :

Traffic volume (commercial vehicles)	Number of vehicles per day
Light	Less than 50
Medium	50 to 300
Heavy	Over 300

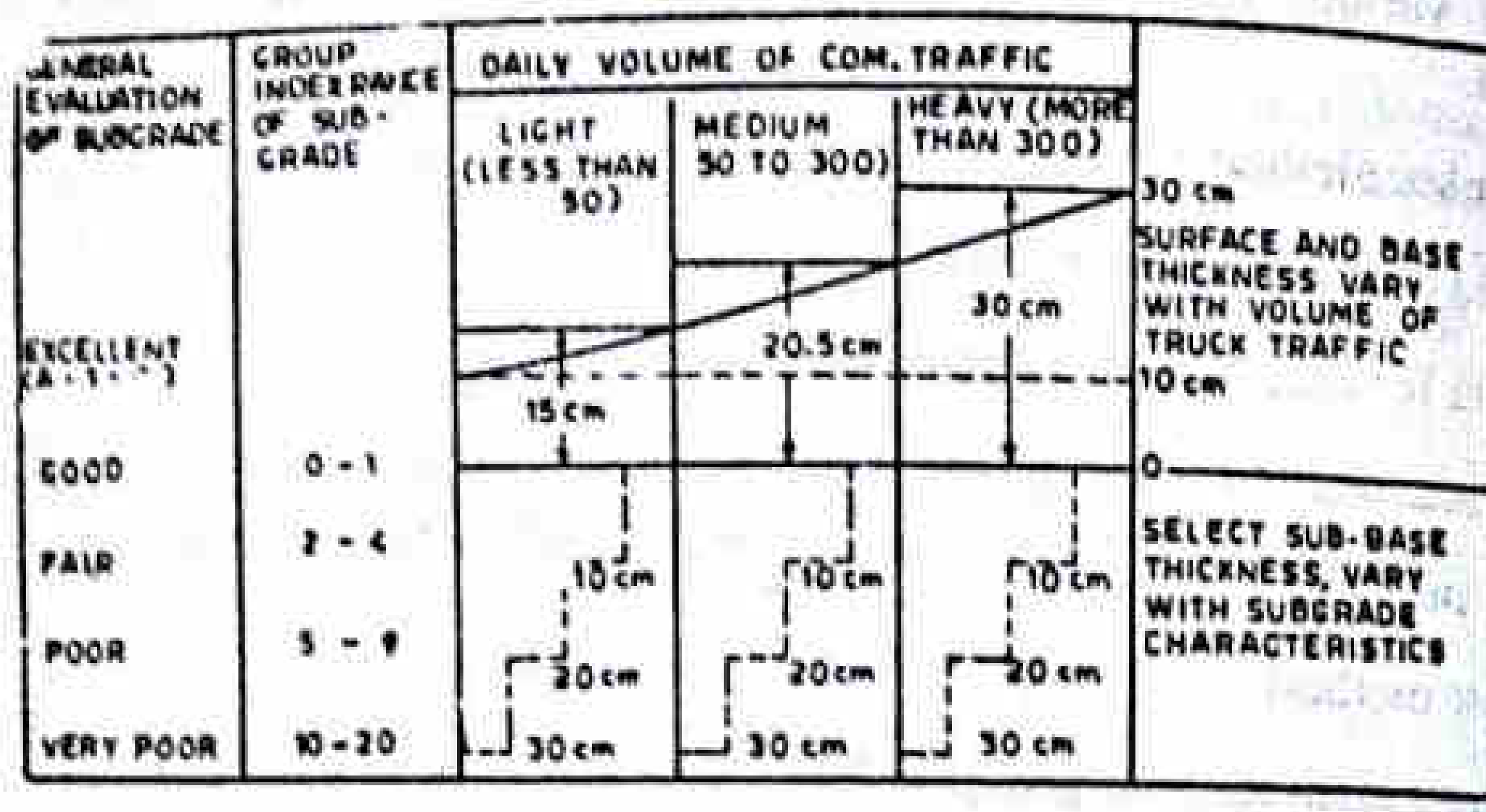
To design the pavement thickness by this method, first the GI value of the soil is found. The anticipated traffic is estimated and is designated as light, medium or heavy as indicated in Fig. 7.9-a. The appropriate design curve is chosen from Fig. 7.9-b and the total thickness of pavement (surface, base and sub-base course) is found from the Group Index design chart corresponding to the GI values of the soil.

Discussion

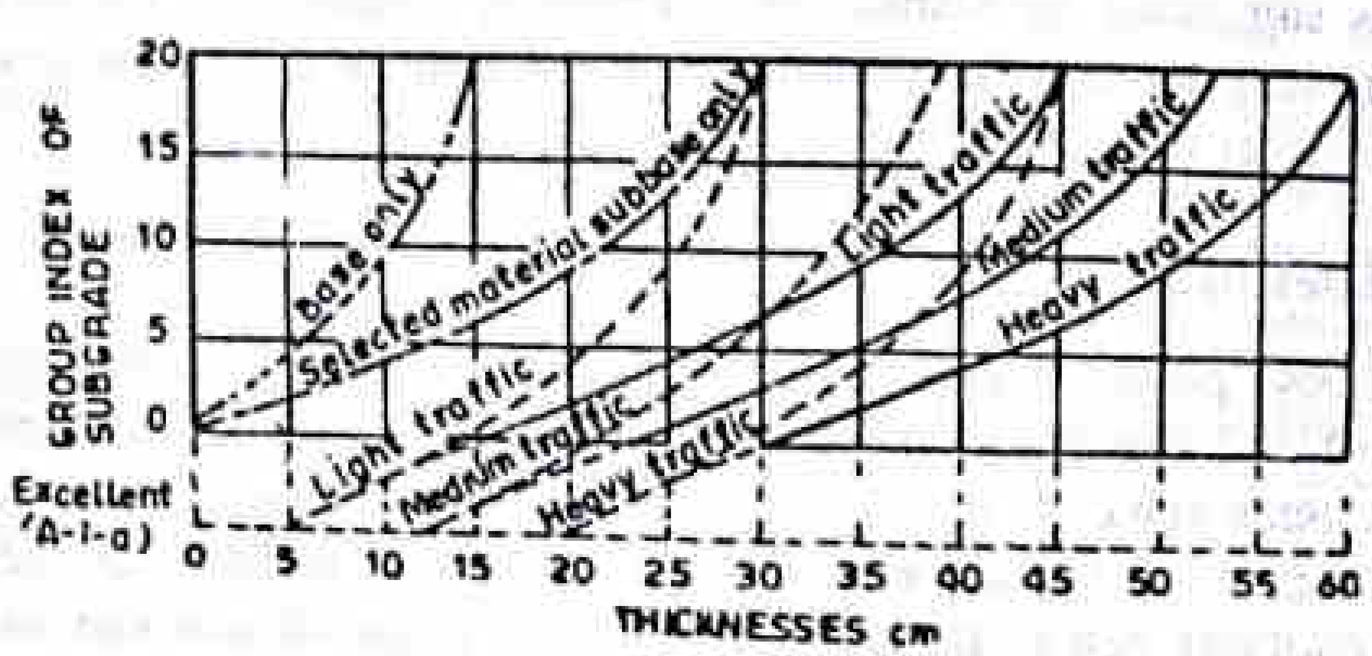
The GI method of pavement design is essentially an empirical method based on physical properties of the subgrade soil. This method does not consider the strength characteristics of the subgrade soil and therefore is open to question regarding the reliability of the design based on the index properties of the soil only. The Group Index method is illustrated by the following example.

Example 7.3

Soil subgrade sample collected from the site was analysed and the results obtained are as given below :



(a)



(b)

— Combined thickness of surface, base and sub-base

--- Thickness of surface and base.

Fig. 7.9 Design Chart by Group Index value

(i) Soil portion passing

0.074 mm sieve, percent = 50

(ii) Liquid Limit, percent = 40

(iii) Plastic Limit, percent = 20

Design the pavement section by group index method for the anticipated traffic volume of over 300 commercial vehicles per day.

Solution

The GI value could be calculated by either using Group Index charts vide Fig. 6.3 or by the Eq. 6.1.

From G I Chart

(i) Numerical value from Chart I for LL = 40 and percent passing 0.074 mm sieve = 50, is equal to 3.

(ii) Plasticity Index = LL − PL = 40 − 20 = 20. Numerical value from Chart II for PI = 20 and percent passing 0.074 mm sieve = 50, is equal to 3.5.

Total of value from Chart I + Chart II = 3 + 3.5 = 6.5 say GI = 7

From G I Equation

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

Here

$$a = 50 - 35 = 15;\ b = 50 - 15 = 35$$

$$c = 40 - 40 = 0;\ d = 20 - 10 = 10$$

$$\text{GI value} = 0.2 \times 15 + 0 + 0.01 \times 35 \times 10 = 3 + 3.5 = 6.5 \text{ say } 7$$

Pavement Thickness Determination

The subgrade soil may be rated as poor from Fig. 7.9 (a) as the G.I. = 7. Traffic volume may be taken as heavy. The pavement layers may be designed either using Fig. 7.9 (a) or using the design chart given in Fig. 7.9 (b).

From Design Chart (Fig. 7.9 b)

(i) Thickness of sub-base for GI of 7 = 17 cm

(ii) Combined thickness of surface, base and sub-base course (using curve D for heavy traffic) = 47 cm

Hence thickness of base and surfacing = 47 − 17 = 30 cm

Discussion

It may be seen here that the quality of sub-base and base course materials is not considered in this method. The strength characteristics of the pavement materials also influence the thickness requirement. In this method, the emphasis is given only on subgrade soil type and certain physical properties of the soil.

California bearing ratio method

In 1928 California Division of Highways in the U.S.A. developed CBR method for pavement design. The majority of design curves developed later are based on the original curves proposed by *O. J. Porter*. At the beginning of the second World War, the *Corps Engineer* of USA made survey of the existing method of pavement design and adopted CBR method for designing military airport pavements. One of the chief advantages of CBR method is the simplicity of the test procedure. Details of CBR tests are explained in article 6.1.5.

The CBR tests were carried out by the California State Highway Department on existing pavement layers including subgrade, sub-base and base course. Based on the extensive CBR test data collected on pavement which behaved satisfactorily and those which failed, an empirical design chart was developed correlating the CBR value and the pavement thickness. The basis of the design chart is that a material with a given CBR required a certain thickness of pavement layer as a cover. A higher load needs a thicker pavement layer to protect the subgrade. Design curves correlating the CBR value with total pavement thickness cover were developed by the California State Highway Department for wheel loads of 3175 kg and 5443 kg representing light and heavy traffic. Later the design curve for 4082 kg wheel load was obtained by interpolation for medium traffic. The design curves are shown in Fig. 7.10.

Studies carried out by U. S. Corps of Engineers have shown that there exists a relationship between pavement thickness, wheel load, tyre pressure and C.B.R., value within a range of 10 to 12 percent. Therefore it is possible to extend the CBR design curves for various loading conditions, using the expression :

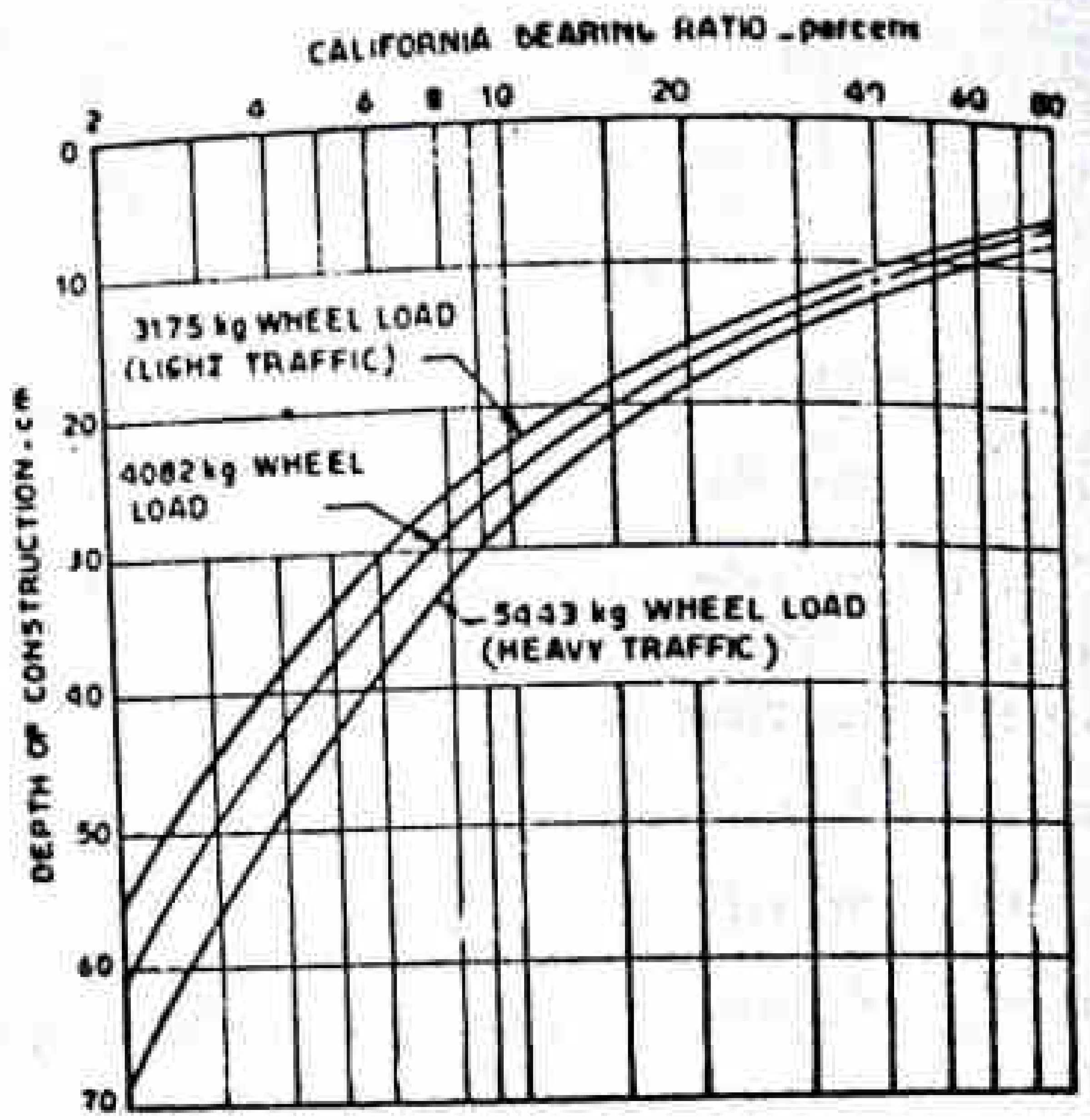


Fig. 7.10 Design Chart (California State Highway Department)

$$t = \sqrt{P}\left[\frac{1.75}{CBR} - \frac{1}{p\pi}\right]^{\frac{1}{2}} \qquad (7.6\text{-a})$$

$$t = \left[\frac{1.75\,P}{CBR} - \frac{A}{\pi}\right]^{\frac{1}{2}} \qquad (7.6\text{-b})$$

However these expressions are applicable only when the CBR value of the subgrade soil is less than 12 percent.

Here, t = pavement thickness, cm

P = wheel load, kg

CBR = California Bearing Ratio, percent

P = tyre pressure, kg/cm^2

A = area of contact, cm^2

The Indian Road Congress has recommended a CBR design chart for tentative use in India. Different curves A, B, C, D, E, F & G have been given based on the volume of commercial vehicles. See Fig. 7.11. This design chart is similar to the one followed in U.K.

Pavement Thickness Determination

In order to design a pavement by CBR method, first the soaked CBR value of the soil subgrade is evaluated. Then the appropriate design curve is chosen by taking the design wheel load as given in Fig. 7.10 or by taking the anticipated traffic into consideration (as given in Fig. 7.11). Thus the total thickness of flexible pavement needed to cover the subgrade of the known CBR value is obtained. In case there is a material superior than the soil subgrade, such that it may be used as sub-base course then the thickness of construction over this material could be obtained from the design chart knowing the CBR value of the sub-base. Thickness of the sub-base course is the total thickness minus the thickness over the sub-case.

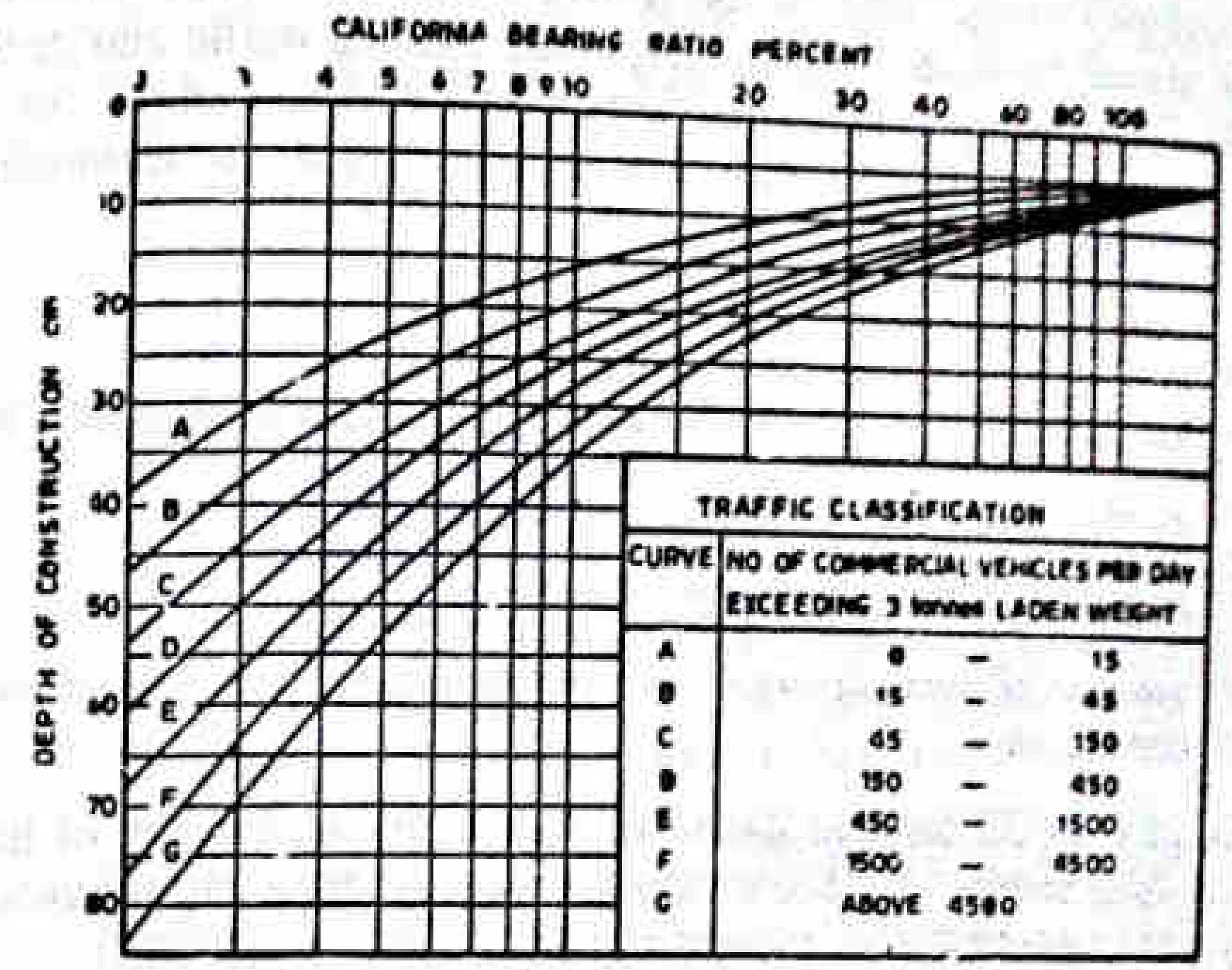


Fig. 7.11 C.B.R. Design Chart (Recommended by IRC)

Thus CBR method of flexible pavement design is based on strength parameter of subgrade soil and subsequent pavement material.

IRC Recommendations

Some of the important points recommended by the IRC for the CBR method of design (IRC : 37-1970) are given below :

(a) The CBR tests should be performed on remoulded soils in the laboratory. In-situ tests are not recommended for design purposes. The specimens should be prepared by static compaction wherever possible and otherwise by dynamic compaction. The standard test procedure should be strictly adhered to.

(b) For the design of new roads, the subgrade soil sample should be compacted at OMC to Proctor density whenever suitable compaction equipment is available to achieve this density in the field; otherwise the soil sample may be compacted to the dry density expected to be achieved in the field. In the case of existing roads, the sample should be compacted to field density of subgrade soil (at OMC or at a field moisture content).

(c) In new constructions the CBR test samples may be soaked in water for four days period before testing. However in areas with arid climate or when the annual rainfall is less than 50 cm and the water table is too deep to affect the subgrade adversely and when thick and impermeable bituminous surfacing is provided, it is not necessary to soak the soil specimen before carrying out CBR test. Wherever possible the most adverse moisture condition of the subgrade should be determined from the field study.

(d) Atleast three samples should be tested on each type of soil at the same density and moisture content. If the maximum variation in CBR values of the three specimens exceeds the specified limits, the design CBR should be the average of at least six samples. (The specified limits of maximum variation in CBR are 3% for CBR values upto 10% 5 for values 10 to 30 and 10% for values 30 to 60%).

(e) The top 50 cm of subgrade should be compacted atleast upto 95 to 100 percent of Proctor density.

(f) An estimate of the traffic to be carried by the road pavements at the end of expected life should be made keeping in view the existing traffic and probable growth rate of traffic. Pavements of major roads should be designed atleast for 10 years life period and the following formula may be used in such cases for estimating the design traffic.

$$A = P[1 + r]^{(n + 10)} \qquad (7.6\text{-c})$$

where A = number of heavy vehicles per day for design (laden weight > 3 tonnes)

P = number of heavy vehicles per day at least count

r = annual rate of increase of heavy vehicles

n = number of years between the last count and the year of completion of construction.

The value of P in the formula should be the seven day average of heavy vehicles found from 24-hour counts. If reliable values of growth factor r is not available, a value of 7.5% may be assumed for roads in rural areas.

(g) The traffic for the design is considered in units of heavy vehicles (of laden weight exceeding 3 tonnes) per day in both directions and are divided into seven categories A to G. The suitable design curve should be chosen from the Table given in the design chart, (Fig. 7.11) after estimating the design traffic given in Eq. 7.6-c. The design thickness is considered applicable for single axle loads *upto* 8,200 kg and tandem axle loads upto 14,500 kg. For higher axle loads, the thickness values should be further increased.

(h) When sub-base course materials contain substantial proportion of aggregates of size above 20 mm, the CBR value of these materials would not be valid for the design of subsequent layers above them. Thin layers of wearing course such as surface dressing or open graded premixed carpet upto 2.5 cm thickness should not be counted towards the total thickness as they do not increase the structural capacity as the pavement.

Example 7.4

The CBR value of subgrade soil is 5%, calculate total thickness of a pavement using

(i) design curve developed by California State Highway Department

(ii) design chart recommended by IRC

(iii) design formula developed by the US Crops of Engineers

Assume 4100 kg wheel load or medium light traffic of 200 commercial vehicles per day for design.

Tyre pressure = 6 kg/cm^2

Solution

(i) Using the design chart of California State Highway Department, the pavement thickness for 4100 kg wheel load and CBR = 5% (See Fig. 7.9) = 38 cm.

(ii) Using the design chart recommended by IRC (see Fig. 7.10) for 200 commercial vehicles per day and using curve D and for CBR value = 5% the thickness = 37.5 cm.

(iii) Using design formula given in Eq. 7.6-a,

$$t = \sqrt{P}\left[\frac{1.75}{CBR} - \frac{1}{p\pi}\right]^{\frac{1}{2}}$$

$$P = 4100 \text{ kg}$$

$$p = 6 \text{ kg/cm}^2$$

$$t = \sqrt{4100}\left[\frac{1.75}{5} - \frac{1}{6\pi}\right]^{\frac{1}{2}} = 35.5 \text{ cm}$$

Example 7.5

Soil subgrade sample was obtained from the project site and the CBR tests was conducted at field density. The following were the results :

Penetration mm	Load kg	Penetration mm	Load kg
0.0	0.0	3.0	56.5
0.5	5.0	4.0	67.5
1.0	16.2	5.0	75.2
1.5	28.1	7.5	89.0
2.0	40.0	10.0	99.5
2.5	48.5	12.5	106.5

It is desired to use the following materials for different pavement layers.

(i) Compacted sandy soil with 7 percent CBR

(ii) Poorly graded gravel with 20 percent CBR

(iii) Well graded gravel with 95 percent CBR

(iv) Minimum thickness of bituminous concrete surfacing may be taken as 5 cm

The traffic survey revealed the present ADT of commercial vehicle as 1200. The annual rate of growth of traffic is found to be 8 percent. The pavement construction is to be completed in three years after the last traffic count.

(a) Design the pavement section by CBR method as recommended by IRC, using all the four pavement materials.

(b) Suggest alternate design without using poorly graded gravel.

Discuss the limitation of CBR method of pavement design in the light of the above results.

Solution

CBR Value of Soil Subgrade

The plot is made between load in kg versus penetration of plunger for the test data obtained for soil subgrade as given in Fig. 7.12. Loads at 2.5 and 5.0 mm penetration (after correction) are 55 and 78 kg respectively.

Area of plunger of dia 5 cm = 19.6 cm^2

Pressure at 2.5 mm penetration = $\frac{55}{19.6}$ kg/cm^2

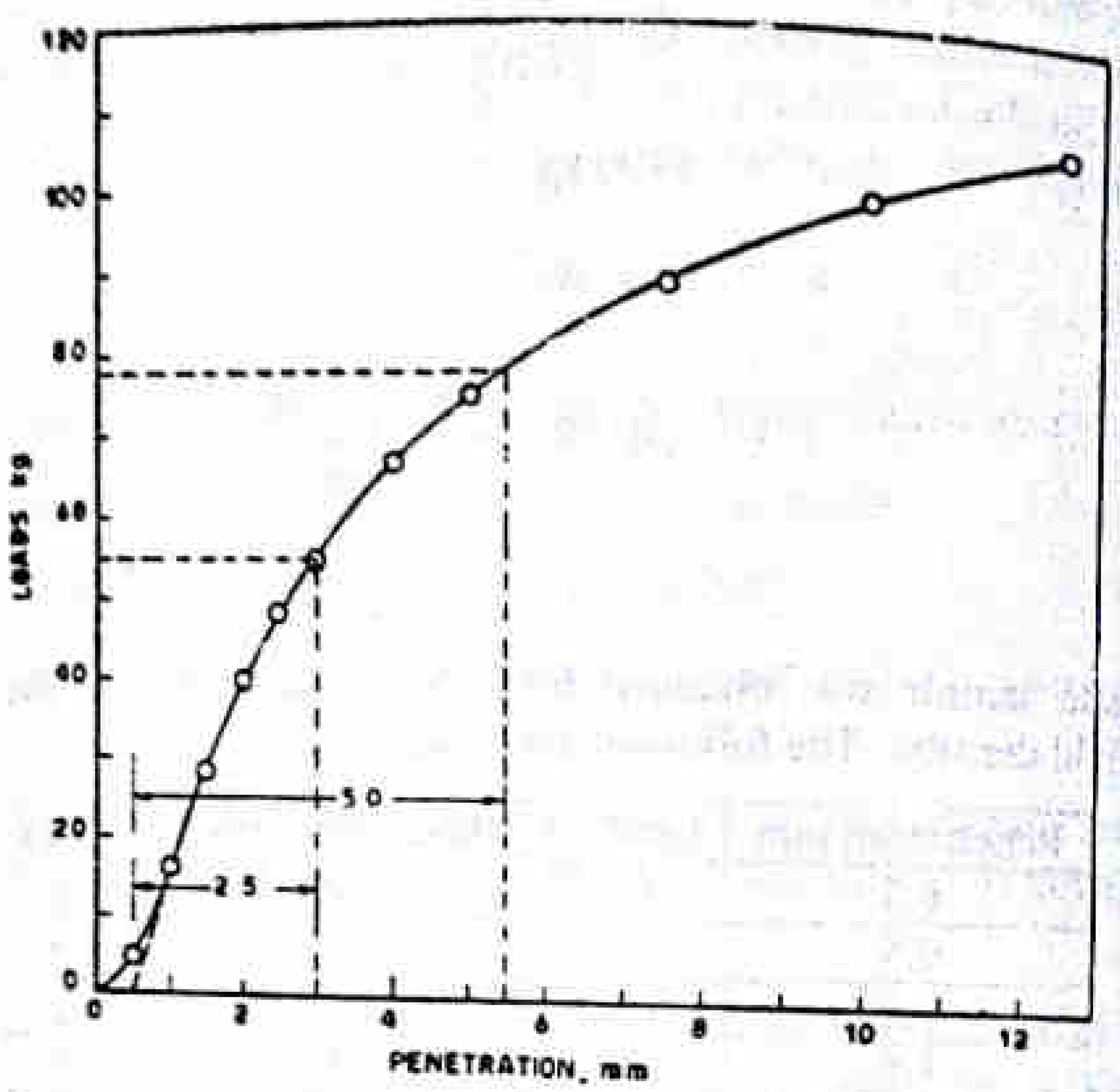


Fig. 7.12 Load-Penetration Curve (Example 7.5)

$$\text{Pressure at 5 mm penetration} = \frac{78}{19.6}\ \text{kg/cm}^2$$

$$\text{C.B.R. value of soil at 2.5 mm} = \frac{\text{Pressure on plunger @ 2.5 mm penetration for soil}}{\text{Pressure as above for standard crushed stones}} \times 100$$

$$= \frac{55}{19.6} \times \frac{100}{70} = 4.0 \text{ percent}$$

$$\text{CBR of Soil at 5 mm } \frac{78 \times 100}{19.6 \times 105} = 3.8 \text{ percent}$$

$$\text{Adopt CBR value} = 4.0 \text{ percent}$$

Calculation of Design Thickness of Different Layers

No. of vehicles for design (from Equation 7.6-c) is given by

$$A = P(1+r)^{(n+10)} = 1200\left[1+\frac{8}{100}\right]^{(3+10)}$$

$$= 3260 \text{ vehicles/day}$$

Therefore Design Curve F is to be used for design as the design traffic volume is in the range 1500 to 4500 cv/day.

Using the design chart vide Fig. 7.11, the total pavement thickness over subgrade having CBR of 4 percent is obtained as 55 cm for curve F.

Thus 55 cm of pavement materials is required to cover the natural soil subgrade having 4% CBR value. Now to compute the thickness of compacted soil, the design

curve D is again used for CBR value of 7 percent. Pavement thickness of 40 cm is required above the compacted soil subgrade having CBR value of 7 percent and hence the actual thickness of this layer is 55 − 40 = 15 cm. Similarly the thickness of pavement required over poorly graded gravel of CBR 20 percent and well graded gravel of CBR 95 percent are 21 cm and 8 cm respectively.

The designed pavement section is shown in Fig. 7.13.

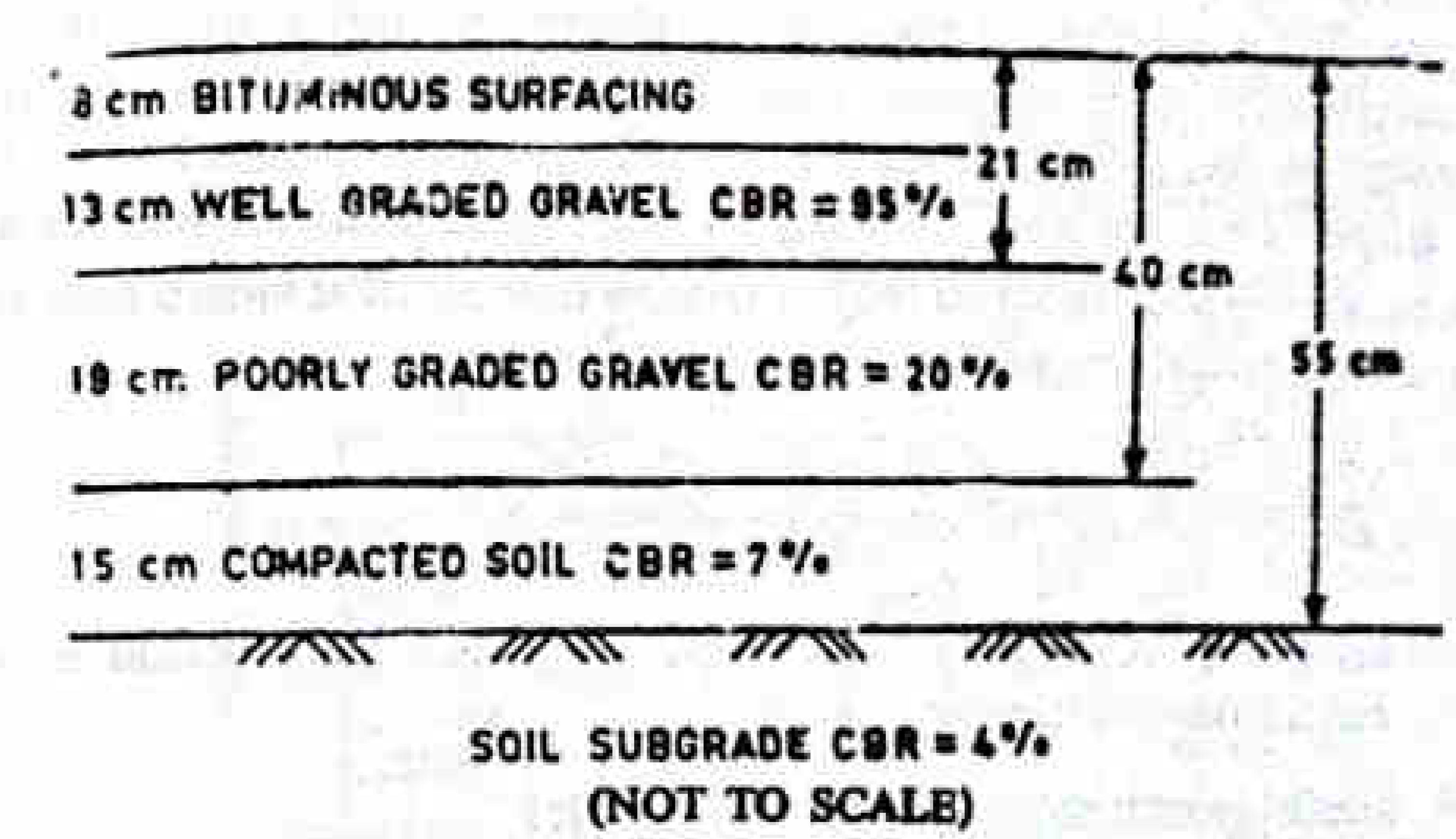


Fig. 7.13 Pavement Section by CBR method (Example 7.5)

Alternatively, if it is considered not to use poorly graded gravel as employed above, then the design section would be as shown in Fig. 7.14.

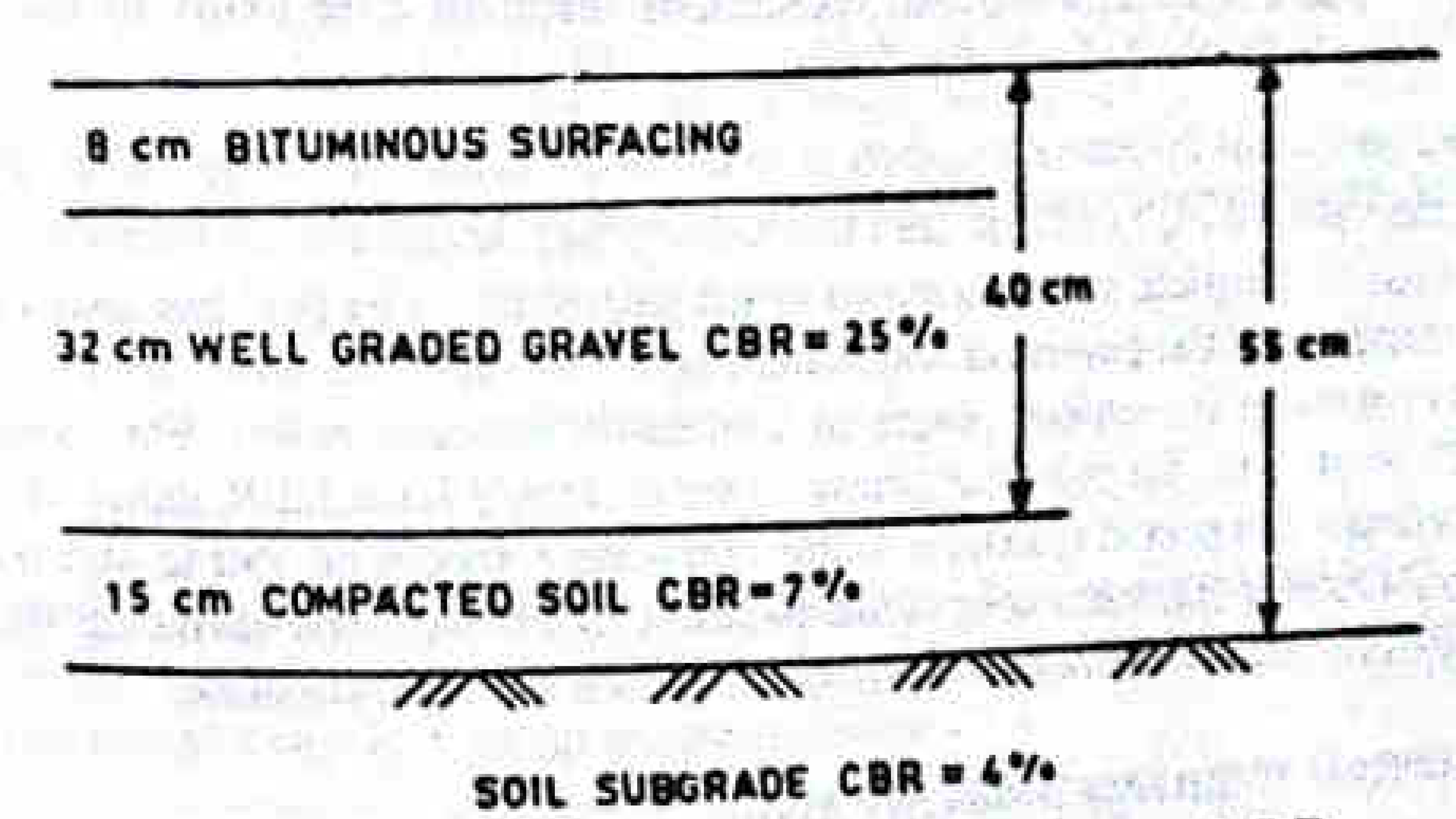


Fig. 7.14 Alternate Pavement Section (Example 7.5)

Discussion on Limitations of the Method

The CBR method suffers from one disadvantage. It may be seen that the total thickness of construction remains same i.e. 55 cm though the pavement component layers are of different materials with different CBR values. The thickness of construction over compacted soil of CBR value 7 percent is same in both cases equal to 40 cm though in one case poorly graded gravel of CBR value 20 percent is used where as in the second case, it has been replaced by well graded gravel of CBR value 95 percent.

Therefore the first proposal is more economical as the thickness of well graded gravel base course is partially replaced by inferior material at the lower layer.

The CBR method of pavement design gives the total thickness requirement of the pavement above a subgrade and this thickness value would remain the same irrespective of the quality of materials used in component layers. Thus the combination of different materials should be judiciously chosen to effect durability and economy of the pavement.

CBR Method of Pavement Design by Cumulative Standard Axle Load

The Indian Roads Congress vide IRC : 37–1984 has revised the guidelines for the Design of Flexible Pavements, based on the concept of Cumulative Standard Axle Loads rather than the total number of all commercial vehicles as done earlier. In the case of roads with design traffic more than 1500 commercial vehicle per day, the design traffic is defined in terms of the cumulative number of standard axle loads of 8160 kg carried during the design life of the road. The mixed commercial vehicles with different axle loads are to be converted in terms of the cumulative number of standard axle load, N_s to cater for the design, using the equation :

$$N_s = \frac{365\,A\,[(1+r)^n - 1]}{r} \times F \qquad (7.7)$$

where A = number of commercial vehicles per day when construction is completed considering the number of lanes.

r = annual growth rate of commercial vehicles

n = design life of pavement, taken as 10 to 15 years

F = vehicle damage factor, equivalent to number of standard axles per commercial vehicle on the road stretch. This is a factor converting the number of commercial vehicles of different axle loads to the number of standard axle load repetitions.

The total pavement thickness required is determined using the design chart given in Fig. 7.15, with the value of N_s in million standard axles (msa) determined as mentioned above and the CBR value of subgrade soil determined in the laboratory. The IRC has also suggested the minimum thickness of the pavement component layers of sub-base, base course and surfacing and the combinations for various ranges of cumulative standard axles. For example for the range of 20 to 30 msa, the sub-base course material should have CBR value of atleast 30% and the minimum compacted thickness of this component should be 390 to 405 mm; the base course should have a minimum compacted thickness of 250 mm and surfacing should consist of 100 to 15 mm dense bituminous macadam and 40 mm asphaltic concrete.

7.3.4 California Resistance Value Method

F. M. Hveem and *R. M. Carmany* in 1948 provided design method based on stabilometer R-value and cohesiometer C-value. The working of the stabilometer and cohesiometer are explained in article 6.4. Based on performance data, it was established by Hveem and Carmany that pavement thickness varies directly with R value and logarithm of load repetitions. It varies inversely with fifth root of C value. The expression for pavement thickness is given by the empirical equation :

$$T = \frac{K\,(TI)\,(90 - R)}{C^{1/5}} \qquad (7.8)$$

Here, T = total thickness of pavement, cm

K = numerical constant = 0.166

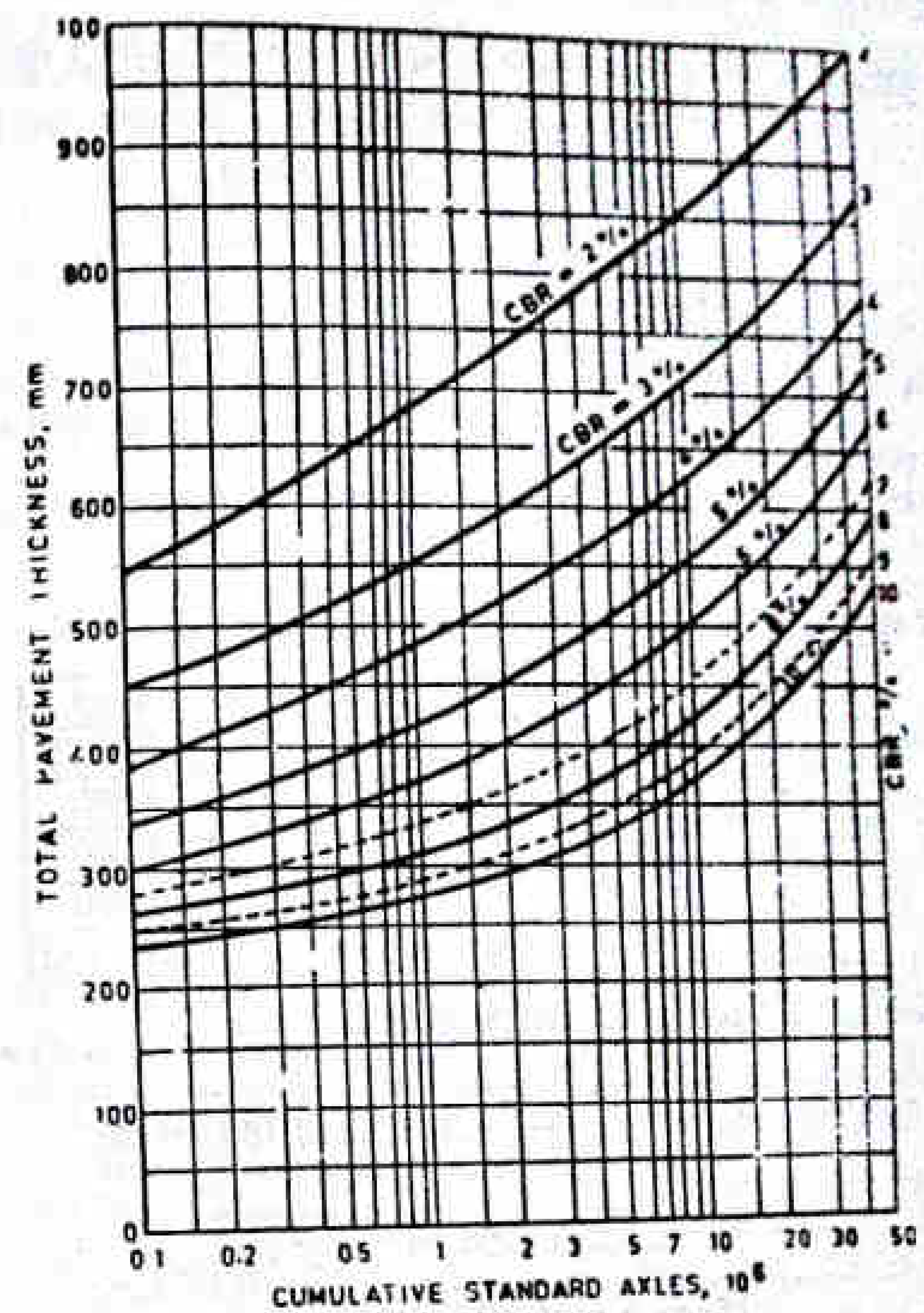


Fig. 7.15 CBR Method of Pavement Design by Cumulative Standard Axle Load

TI = traffic index = 1.35 $(EWL)^{0.11}$ (7.8a)

R = stabilometer resistance value

C = cohesiometer value

The annual value of equivalent wheel load (EWL) here is the accumulated sum of the products of the constants and the number of axle loads. The various constants for the different number of axles in a group are given below :

Number of axles	EWL constants (yearly basis)
2	330
3	1070
4	2460
5	4620
6	3040

These contents were obtained based on the State wide *loadometer survey* carried out in California during 1955-56. Hence if the annual average daily traffic volumes (AADT) data are available for different groups of axles, the yearly EWL is obtained by multiplying by the appropriate constant given above and taking the sum. Method of finding the EWL and traffic index TI has been illustrated in Example 7.6.

Example 7.6

Calculate ten-year EWL and TI values using the following AADT data

Number of axle	AADT (Two directions)
2	3500
3	344
4	295
5	80

Assume 50 percent increase in traffic in 10 year period.

Solution

The product-sum of EWL is calculated as given below :

No. of axles	AADT	EWL constant	Product
2	3500	330	1,15,5000
3	344	1070	368,080
4	295	2460	725,700
5	80	4620	369,600
Total yearly EWL = 2,618,380			

Taking the average increase for 10-year period then

$$EWL_{10} = \left(\frac{1+1.5}{2}\right) 10 \times 2{,}618{,}380$$

$$= 32{,}729{,}750$$

$$TI = 1.35\,(EWL)^{0.11}$$

$$= 1.35 \times (32{,}729{,}750)^{0.11}$$

$$= 9.057$$

In the design of flexible pavements based on California Resistance value method therefore the following data are needed :

(i) R-value of soil subgrade

(ii) TI value

(iii) Equivalent C-value of pavement materials

R value of soil subgrade is obtained from the test using stabilometer as explained in Art. 6.4 and Eq. 6.17. The computation of TI value has been explained above.

Equivalent C Value

The cohesiometer value C, is obtained for each layer of pavement material separately from tests. It is not possible to have a composite C-value for the total pavement section experimentally. However the composite or equivalent C-value of the pavement may be estimated if the thickness of each component layer and the C-value of the material of the layer is known. The method of calculating the equivalent C-value of a multilayered pavement is illustrated in Example 7.7.

While designing a pavement as the thickness of the pavement is not known, it is easier if the pavement is first assumed to consist of any one material like gravel base course with known C-value. Subsequently the individual thickness of each layer is converted in terms of gravel equivalent by using relationship :

$$\frac{t_1}{t_2} = \left(\frac{C_2}{C_1}\right)^{1/5} \tag{7.9}$$

where, t_1 and t_2 are the thickness values of any two pavement layers and C_1 and C_2 are their corresponding cohesiometer values.

Typical C-values for some pavement materials are given below (in metric equivalents) :

Materials	C-value
Soil-cement base course	120-230
Bituminous concrete	60-62
Open graded bituminous mix	22-30
Gravel base course	15

Example 7.7

Calculate the equivalent C-value of a three layered pavement section having individual C-values as given below :

Materials	Thickness, cm	C-value
Bituminous concrete	10	60
Cement treated base	20	225
Gravel sub-base	10	15

Solution

The individual thickness of each layer is converted to their respective gravel equivalent using the following relationship :

$$\frac{t_g}{t} = \left(\frac{C}{C_g}\right)^{1/5}$$

Here, t_g = gravel thickness

t = individual thickness

C_g = cohesiometer value of gravel = 15

C = respective C-value

$$t_g = \left(\frac{60}{15}\right)^{\frac{1}{5}} \times 10 = 13.2 \text{ cm}$$

For base course, $t_g = \left(\frac{225}{15}\right)^{\frac{1}{5}} \times 20 = 34.4$ cm

For sub-base course, t_g = 10.0 cm

Therefore actual pavement thickness = 10 + 20 + 10 = 40 cm

This is equivalent to gravel thickness = 13.2 + 34.4 + 10.0 = 57.6 cm

Now,
$$\frac{t_g}{T} = \left(\frac{C}{C_g}\right)^{1/5}$$

$$C = \left(\frac{t_g}{T}\right)^5 \times C_g = \left(\frac{57.60}{40}\right)^5 \times 15 = 93$$

The equivalent C-value of the pavement section is 93,

Design Procedure

In this design method it is required to provide a pavement section which satisfies :

(i) Resistance value of sub-grade (R-value)

(ii) Expansion pressure

(iii) Exudation pressure

Laboratory tests are carried out on subgrade soil sample compacted at different moisture contents to find Hveem stabilometer R-values expansion pressure and exudation pressures. The pressure required (applied at rate of about 900 kg per minute) to force out water from a compacted subgrade soil sample is known as exudation pressure and this depends on soil type and the moisture content. As the compacting moisture content of the soil is increased, the R value, exudation and expansion pressure decreases.

In pavement design problems, first the pavement thickness required may be calculated assuming it to consist of a single layer material of known C-value such as gravel or water bound macadam (WBM) base course. Subsequently the thickness of the other component layers are chosen as per the traffic and climatic requirements and the equivalent base course layer thickness to be replaced by these pavement layers are calculated based on their C-values using Equation 7.9.

Design steps

(i) The pavement thickness values required as per the R-values of subgrade soil at different moisture contents are calculated (say, T_{r1}, T_{r2} ...) using Equation 7.8. Here the pavement may first be assumed to consist of single base course layer of known C-value, C_g.

(ii) The pavement thickness values required to counteract the subgrade expansion pressure are found by dividing the expansion pressure by the average density of the pavement which may be assumed as about 2.1 g/cm^2. The pavement thickness value (say Te_1, Te_2 ...) as per expansion pressures at different moisture contents are calculated.

(iii) The pavement thickness fulfilling both R-value and expansion pressure is found by plotting T_r values against the corresponding T_e values from (i) and (ii) above, to the same scale and by drawing a 45° line so that $T_e = T_e$.

(iv) The exudation pressure of subgrade soil found at various compacting moisture contents are plotted against the pavement thickness found from (i) above based on the corresponding R values. The pavement thickness corresponding to an exudation pressure of 28 kg/cm^2 is obtained from this graph, say Td.

(v) The pavement thickness as per California design method is the higher of the values determined in (iii) and (iv) above.

(vi) The thickness of other pavement layers are decided and the equivalent values of base course thickness replaced are calculated using Eq. 7.9 with the known Cohesiometer values of the materials.

The California R-values method of pavement design is a purely empirical method and therefore the test procedure and the specifications should be strictly followed. Nomograms are also available to simplify the design calculations.

The pavement design method is illustrated in Example 7.8.

Example 7.8

Design a flexible-pavement consisting of water bound macadam (WBM) base course and bituminous concrete surface course of thickness 7.5 cm by California R-value (Stabilometer) method using the following data :

Moisture content %	R value	Pressure, kg/cm^2	
		Expansion	Exudation
15	56	0.135	46.5
18	44	0.099	41.5
21	25	0.055	30.5
24	14	0.034	21.5

(i) Test results on subgrade soil

(ii) C-value of WBM base course = 15

(iii) C-value of bituminous concrete surface course = 62

(iv) Traffic index = 9.5

Solution

First find the pavement thickness required using WBM base material only (C value = 15).

(i) Thickness by R-values

$$T = \frac{K\,(TI)(90-R)}{C^{1/5}} = \frac{0.166 \times 9.5 \times (90-56)}{15^{1/5}} = 31.2 \text{ cm}$$

Similarly for R-values of 44, 25 and 14 the pavement thickness values are 42.2, 59.6 and 69.7 cm respectively.

(ii) Thickness by Expansion Pressure

Assuming average pavement density as 2.1 g/cm^3 or 0.0021 kg/cm^3, the pavement thickness needed for counteracting expansion pressure of 0.135 kg/cm^2 = $\frac{0.135}{0.0021}$ = 64.3 cm. Similarly, pavement thickness for expansion pressures of 0.099, 0.055 and 0.034 kg/cm^2 are 47.1, 26.2 and 16.2 cm respectively.

The pavement thickness values obtained by R-values and expansion pressure values at various subgrade moisture contents along with the corresponding exudation pressure are given in the Table below :

Moisture content %	R value	Pressure, kg/cm^2	
		Expansion	Exudation
15	56	0.135	36.5
18	44	0.099	26.5
21	25	0.055	18.0
24	14	0.034	15.0

The pavement thickness values given in this table are plotted in Fig. 7.16 and the equal thickness value by the two methods is obtained by drawing a 45° line. The thickness obtained in this case is $T_r = T_e = 44.5$ cm.

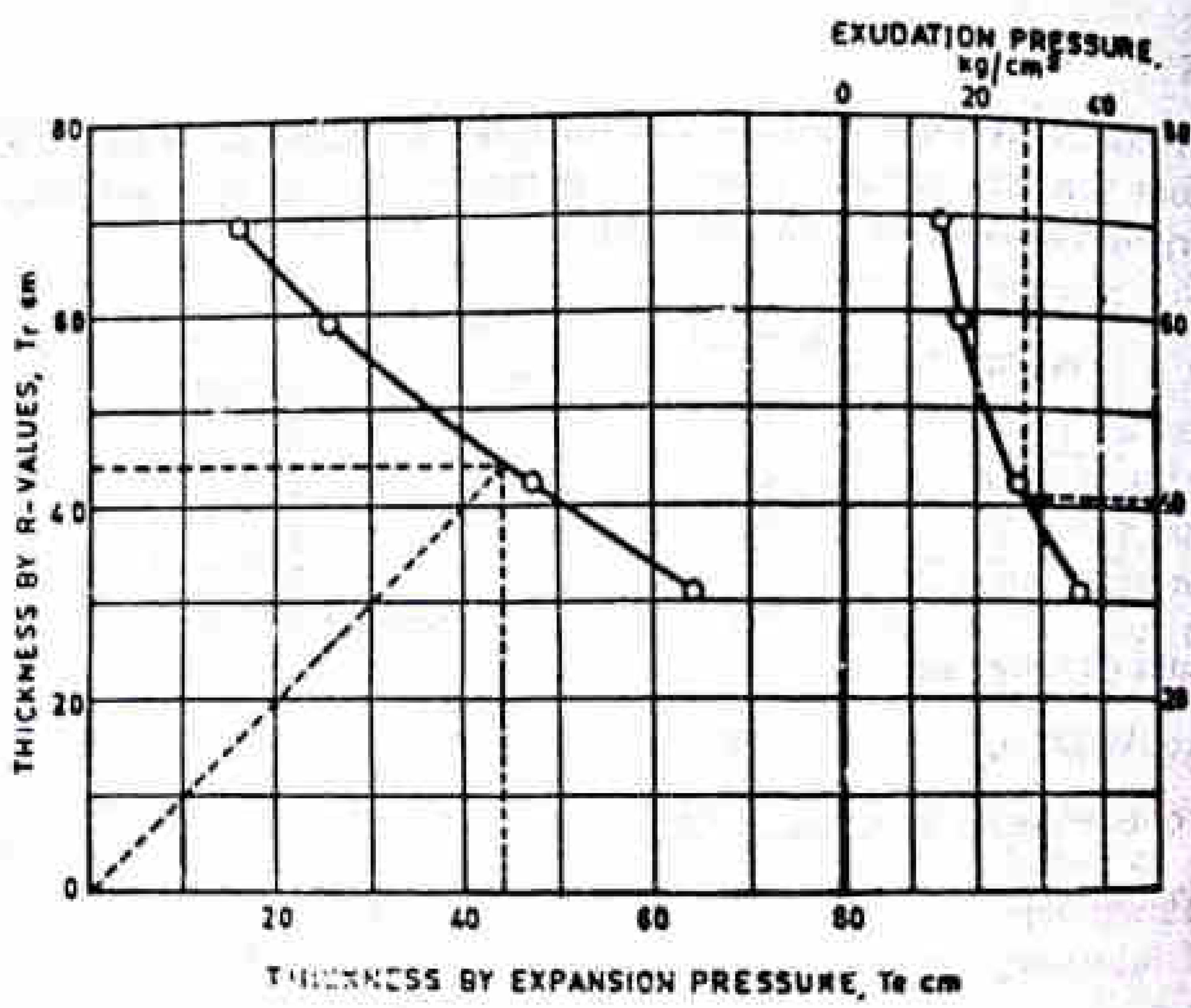


Fig. 7.16 Design of Pavement by R-value Method (Example 7.8)

(iii) Figure 7.16 also shows the plot between the exudation pressure values at the four moisture contents and the corresponding values of pavement thickness calculated based on R-values at these moisture contents. The pavement thickness corresponding to an exudation pressure of 28 kg/cm^2 obtained is 40.5 cm from this plot.

(iv) Therefore the design thickness of single layer WBM pavement is higher of the two values (44.5) and (40.5 cm) and is equal to 44.5 cm.

(v) Bituminous concrete surface course thickness $t_c = 7.5$ cm.

C-value of bituminous concrete $C_c = 62$

C-value of WBM base course $C_b = 15$

Equivalent thickness of 7.5 cm bituminous concrete in terms of WBM base course t_b is given by the relation

$$\frac{t_b}{t_c} = \left(\frac{C_c}{C_b}\right)^{1/5} \quad \text{i.e.,} \quad \frac{t_b}{7.5} = \left(\frac{62}{15}\right)^{1/5}$$

$$t_b = 7.5\left(\frac{62}{15}\right)^{1/5} = 10.0 \text{ cm}$$

WBM base course thickness required = 44.5 – 10.0 = 34.5 cm

Therefore the pavement section consists of 34.5 cm of WBM base course and 7.5 cm of bituminous concrete. The designed pavement section is shown in Fig. 7.17.

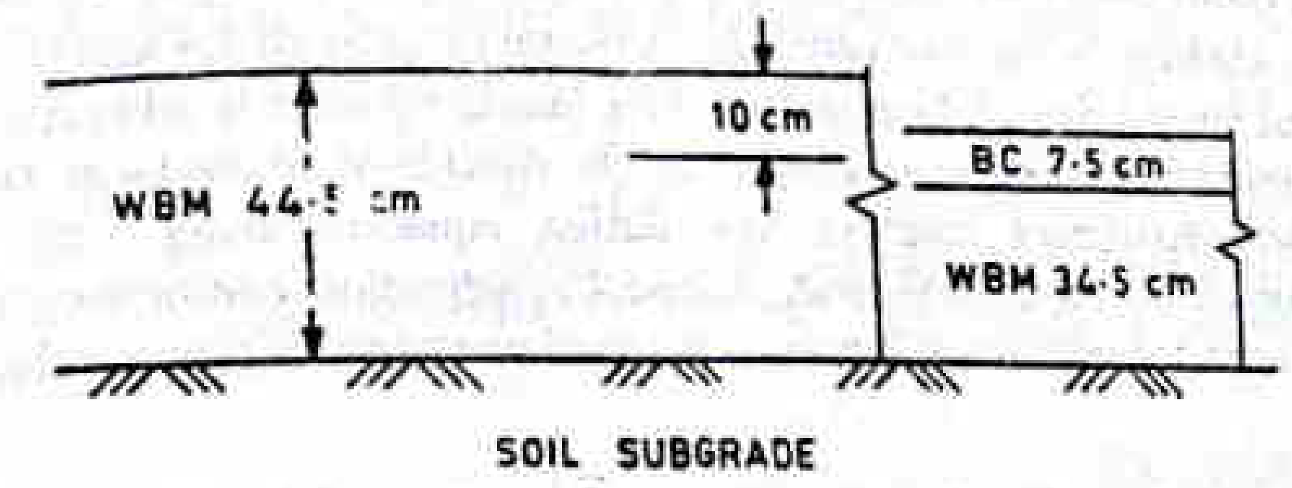


Fig. 7.17 Pavement Section (Example 7.8)

7.3.5 Triaxial Method

L. A. Palmer and *E. S. Barber* in 1910 proposed the design method based on Boussinesq's displacement equation for homogeneous elastic single layer:

$$\Delta = \frac{3\,pa^2}{2E\,(a^2+z^2)^{1/2}} \tag{7.10}$$

Here $\quad p = P/\pi a^2$

$$\therefore \Delta = \frac{3P}{2\pi E\,(a^2+z^2)^{1/2}}$$

$$(a^2+z^2)^{1/2} = \frac{3P}{2\pi E\,\Delta}$$

$$(a^2+z^2) = \left(\frac{3P}{2\pi E\,\Delta}\right)^2$$

$$z = \sqrt{\left(\frac{3P}{2\pi E\,\Delta}\right)^2 - a^2}$$

Assuming that the pavement is incompressible, z becomes T, the thickness of pavement.

$$T = \sqrt{\left(\frac{3P}{2\pi E_s\,\Delta}\right)^2 - a^2} \tag{7.11}$$

Here T = pavement thickness, cm

P = wheel load, kg

E_s = modulus of elasticity of subgrade from triaxial test results, kg/cm^2.

a = radius of contact area, cm

Δ = design deflection (0.25 cm)

In the above analysis the pavement and the subgrade are assumed to have the same E-value.

Use of Triaxial Test

The triaxial compression test as explained in article 6.1.8 is used in determining the values of elastic modulii for various materials. A lateral pressure of 1.4 kg/cm^2 is applied in the test to find the E value of the material. This lateral pressure is arbitrarily assumed as the lateral confinement in pavement layers by the *Kansas State Highway Department* of USA. This department employs this design equation along with empirical modifications for : (i) traffic coefficient, X and (ii) saturation coefficient, Y. These coefficients are used as multiplying factors to the total pavement thickness value which is thus modified.

The pavement thickness T_s consisting of material with modulus E_s is given by the equation :

$$T_s = \sqrt{\left(\frac{3P\,XY}{2\pi E_s \Delta}\right)^2 - a^2} \qquad (7.12)$$

The recommended values of coefficients X and Y based on ADT of design traffic and rainfall are given below :

Traffic coefficient (X)	ADT (number)
1/2	40 – 400
2/3	401 – 800
5/6	801 – 1200
1	1201 – 1800
7/6	1801 – 2700
8/6	2701 – 4000
9/6	4001 – 6000
10/6	6001 – 9000
11/6	9001 – 13,500
12/6	13501 – 20,000

Rainfall coefficient (Y)	Average annual rainfall, cm
0.5	38 – 50
0.6	51 – 64
0.7	65 – 76
0.8	77 – 90
0.9	91 – 100
1.0	101 – 127

If pavement and subgrade are considered as a two layer system, a *Stiffness factor* has to be introduced to take into account the different values of modulus of elasticity of the two layers. The pavement thickness is then modified using the stiffness factor equal to $(E_s/E_p)^{1/3}$ where E_s and E_p are values of modulus of elasticity of the subgrade and pavement, respectively. Thus the thickness of pavement, T_p is calculated from the relation :

$$T_p = \left\{\sqrt{\left(\frac{3P\,XY}{2\pi E_s \Delta}\right)^2 - a^2}\right\}\left(\frac{E_s}{E_p}\right)^{1/3} \qquad (7.13)$$

The thickness design equation 7.11 is based on elastic theory. However the modified equations taking into account the traffic and rainfall coefficients and stiffness factor (Equations 7.12 and 7.13) are empirical modifications. The relation between pavement layers of thickness t_1 and t_2 of elastic modulus E_1 and E_2 is given by :

$$\frac{t_1}{t_2} = \left(\frac{E_2}{E_1}\right)^{1/3}$$

Thus the Kansas Highway Department design method may be categorized as semi-theoretical or semi-empirical method, using triaxial test results.

Example 7.9

Design the pavement section by triaxial test method using the following data :

Wheel load = 4100 kg

Radius of contact area = 15 cm

Traffic coefficient, X = 1.5

Rainfall coefficient Y = 0.9

Design deflection Δ = 0.25 cm

E-value of subgrade soil E_s = 100 kg/cm^2

E-value of base course material E_b = 400 kg/cm^2

E-value of 7.5 cm thick bituminous concrete surface course = 1000 kg/cm^2

Solution

Assuming the pavement to consist of single layer of base course material only; the pavement thickness is given by :

$$T_b = \left\{\sqrt{\left(\frac{3P\,XY}{2\pi E_s \Delta}\right)^2 - a^2}\right\}\left(\frac{E_s}{E_b}\right)^{1/3}$$

$$= \left\{\sqrt{\left(\frac{3\times 4100\times 1.5\times 0.9}{2\pi\times 100\times 0.25}\right)^2 - 15^2}\right\}\left(\frac{100}{400}\right)^{\frac{1}{3}} = 104.64\times 0.63 = 65.9 \text{ cm}$$

Let 7.5 cm bituminous concrete surface with E_c = 1000 kg/cm^2 be equivalent to the thickness t_b of base course. The equivalent replacement t_b is obtained from the relation :

$$\frac{t_b}{t_c} = \left(\frac{E_c}{E_b}\right)^{1/3} \quad \text{i.e., } \frac{t_b}{7.5} = \left(\frac{1000}{400}\right)^{1/3}$$

$$t_b = 7.5\times\left(\frac{1000}{400}\right)^{1/3} = 10.2 \text{ cm}$$

Therefore the required base course thickness = 65.9 – 10.2 = 55.7 cm

The pavement section consists of 55.7 cm thick WBM base course and 7.5 cm thick bituminous concrete surface course. See Fig. 7.18.

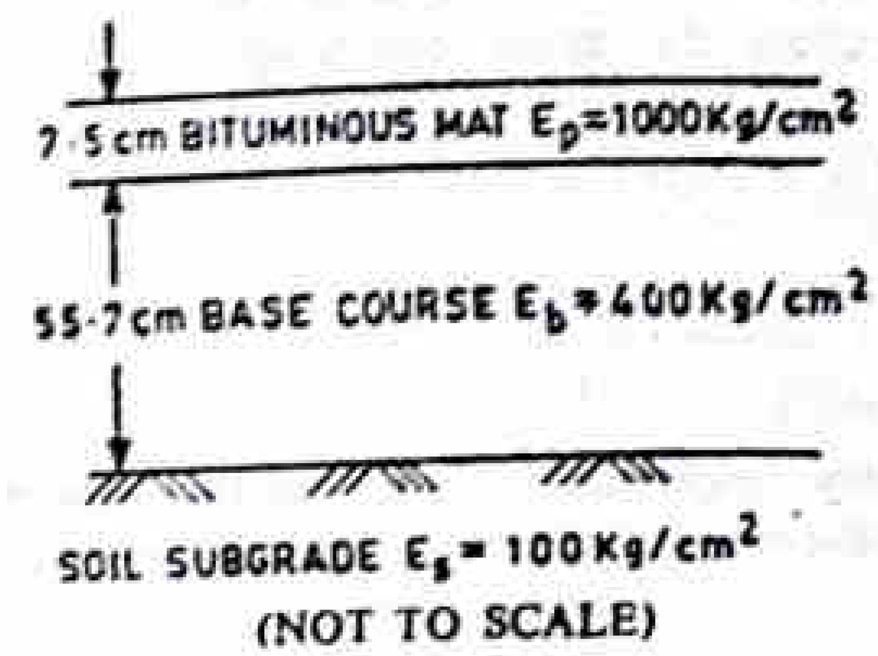


Fig. 7.18 Pavement Section with Base Course

7.3.6 McLeod Method

Norman W. McLeod through Canadian Department of Transport conducted extensive plate bearing tests on airfield and highway pavements and developed a design method. The repetitive plate bearing test procedure was employed using various sizes of plates.

From the plate load tests an empirical design equation was recommended :

$$T = K \log_{10} \frac{P}{S} \tag{7.14}$$

Here T = required thickness of gravel base, cm

P = gross wheel load, kg

= total subgrade support, kg (for the same contact area, deflection and number of repetitions of load P)

K = base course constant.

It is found that the base course constant K depends on the loaded area. Figure 7.19 shows the relationship between the plate diameter and base course constant. Thus the granular base course thickness requirement T may be calculated from Eq. 7.14 for a given wheel load P if the subgrade support is known from plate bearing test data. The subgrade support S for the design of highway pavement is calculated from the support measured or calculated for 30 cm diameter plate at 0.5 cm deflection and ten repetitions. Figure 7.20 is used for finding the ratio of unit subgrade support for the design wheel load diameter to that on 30 cm diameter plate at 0.5 cm deflection. The design unit subgrade support is obtained by multiplying the contact pressure of the design load by the above ratio. The total subgrade support S is calculated by multiplying the unit support by the contact area. The design method is illustrated in Example 7.10.

Example 7.10

Design a highway pavement for a wheel load of 4100 kg with a tyre pressure of 5 kg/cm² by McLeod method. The plate bearing test carried out on subgrade soil using 30 cm diameter plate yielded a pressure of 2.5 kg/cm² after 10 repetitions of load at 0.5 cm deflection.

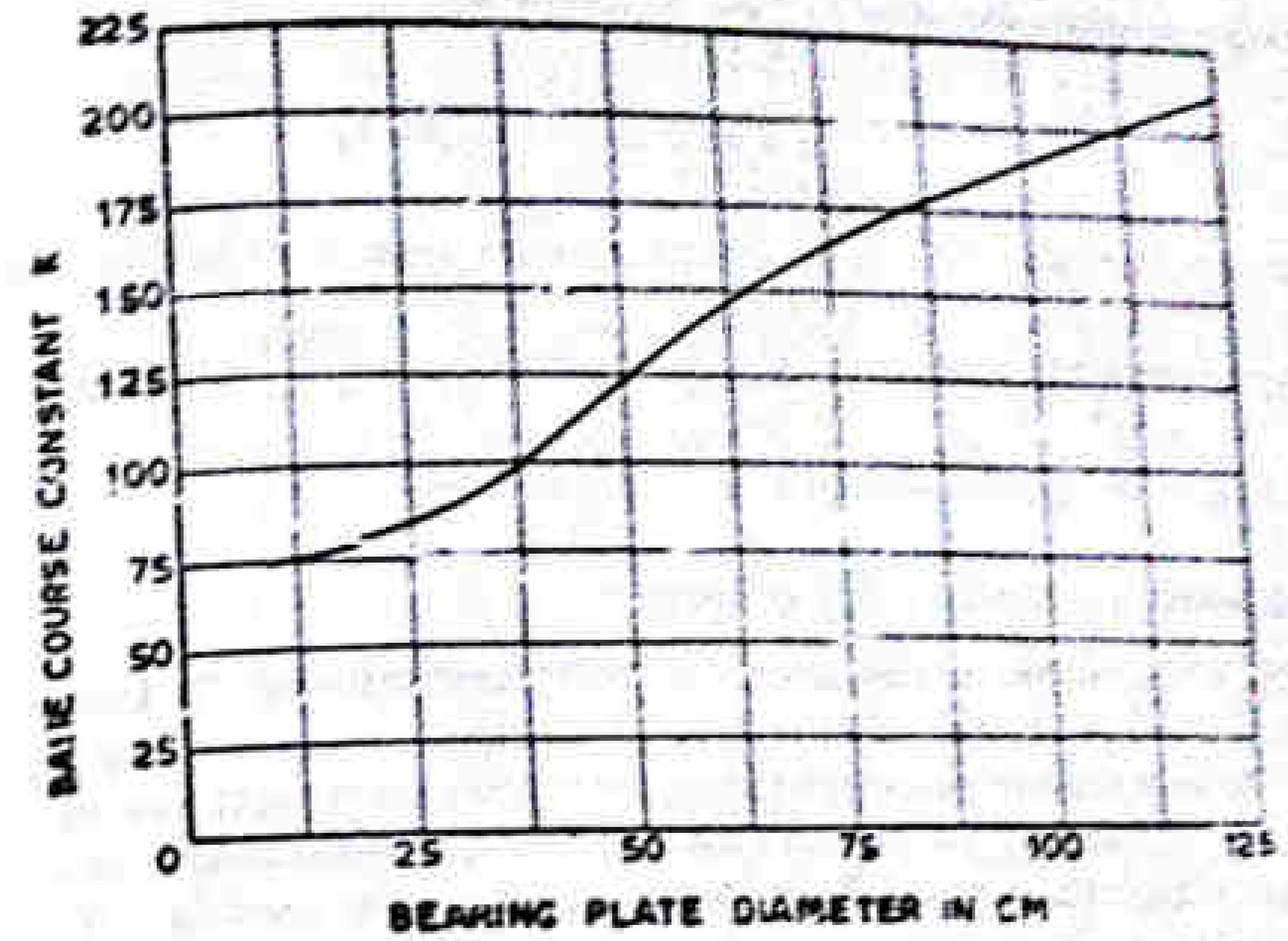


Fig. 7.19 Relation between Plate Diameter and Base Course Constant

Solution

Radius of contact, $a = \sqrt{\frac{P}{p\pi}} = \sqrt{\frac{4100}{5\pi}} = 16.1$ cm

Perimeter over area ratio, $\frac{P}{A} = \frac{2}{a} = \frac{2}{16.1} = 0.124$

Using Fig. 7.20 the ratio of unit subgrade support on 32.2 cm diameter plate at 0.5 cm deflection is 0.95.

Therefore, unit support at 0.5 cm deflection

$$= 0.95 \times 2.5 = 2.44 \text{ kg/cm}^2$$

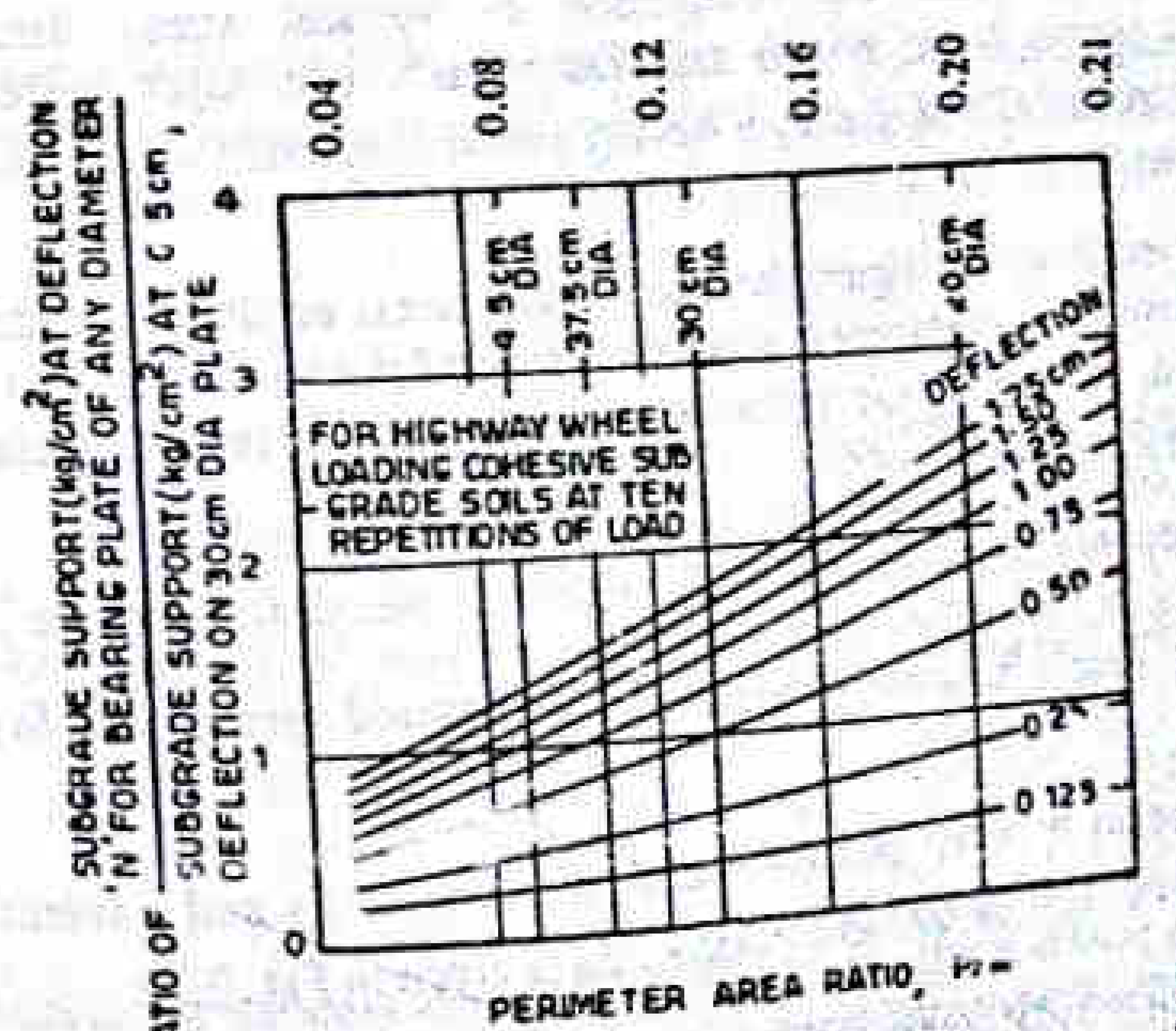


Fig. 7.20 Relationship of Subgrade Support with P/A ratio

Design subgrade support on 32.2 cm diameter plate,

$$S = 2.44\,\pi\,\frac{32.2^2}{4} = 2100 \text{ kg}$$

Base course constant for 32.2 cm diameter plate is obtained from Fig. 7.19 as 90.

$$\text{Granular pavement thickness, } T = K \log_{10}\frac{P}{S} = 90 \log_{10}\frac{4100}{2100} = 26 \text{ cm}$$

Provide 5 cm of bituminous surfacing out of this thickness.

7.3.7 Burmister's (Layered System) Method

Donald M. Burmister developed the layered system analysis. As known the flexible pavement sections are composed of layers and the elastic modulus of the top layer is the highest. The total mass of pavement and subgrade does not possess a constant E value as assumed by Boussinesq in his analysis. However, Boussineq's analysis can be considered as a special case of Burmister's layered system analysis. If layers of soil subgrade, sub-base course and base course are assigned elastic modulii of E_s, E_{sb}, E_b then as per Boussineq's analysis, it is considered $E_s = E_{sb} = E_b$ whereas in layered analysis, it is taken that $E_b > E_{sb} > E_s$. The effectiveness of the reinforcing action of the pavement layers is logically utilised in Burmister's approach. Following are the assumptions made in this approach :

(i) the materials, in the pavement layers are isotropic, homogeneous and elastic. The pavement forms a stiffer reinforcing layer having modulus of elasticity higher than that of the underlying subgrade in the two layer system.

(ii) the surface layer is infinite in horizontal direction and finite in vertical direction; the underlying layer in two layered system is considered infinite in both directions.

(iii) the layers are in continuous contact; the top layer is free of shearing and normal stresses outside the loaded area.

Figure 7.21 provides the comparison of vertical stress distribution between Boussinesq's single layer system and Burmister's two layer system, assuming the pavement to consist of a single layer having elastic modulus E_p lying over subgrade with elastic modulus Es.

It is observed from this figure that the vertical stress on the subgrade is reduced from 70 to 30 percent by introducing a pavement layer of thickness equal to the radius of the load or h = a, having elastic modulus 10 times higher than the elastic modulus of subgrade soil i.e., for $E_p/E_s = 10$.

The Burmister's approach therefore utilises the reinforcing action of the pavement layer.

The deflection factor F_2 is introduced in two layered system which is dependent on E_s/E_p and h/a.

The relationship between two layer deflection factor F_2 and pavement thickness in terms of radius a of loaded area and ratios E_s/E_p is given in Fig. 7.22.

The displacement equations given by Burmister (Equations 7.4 and 7.5) are written here :

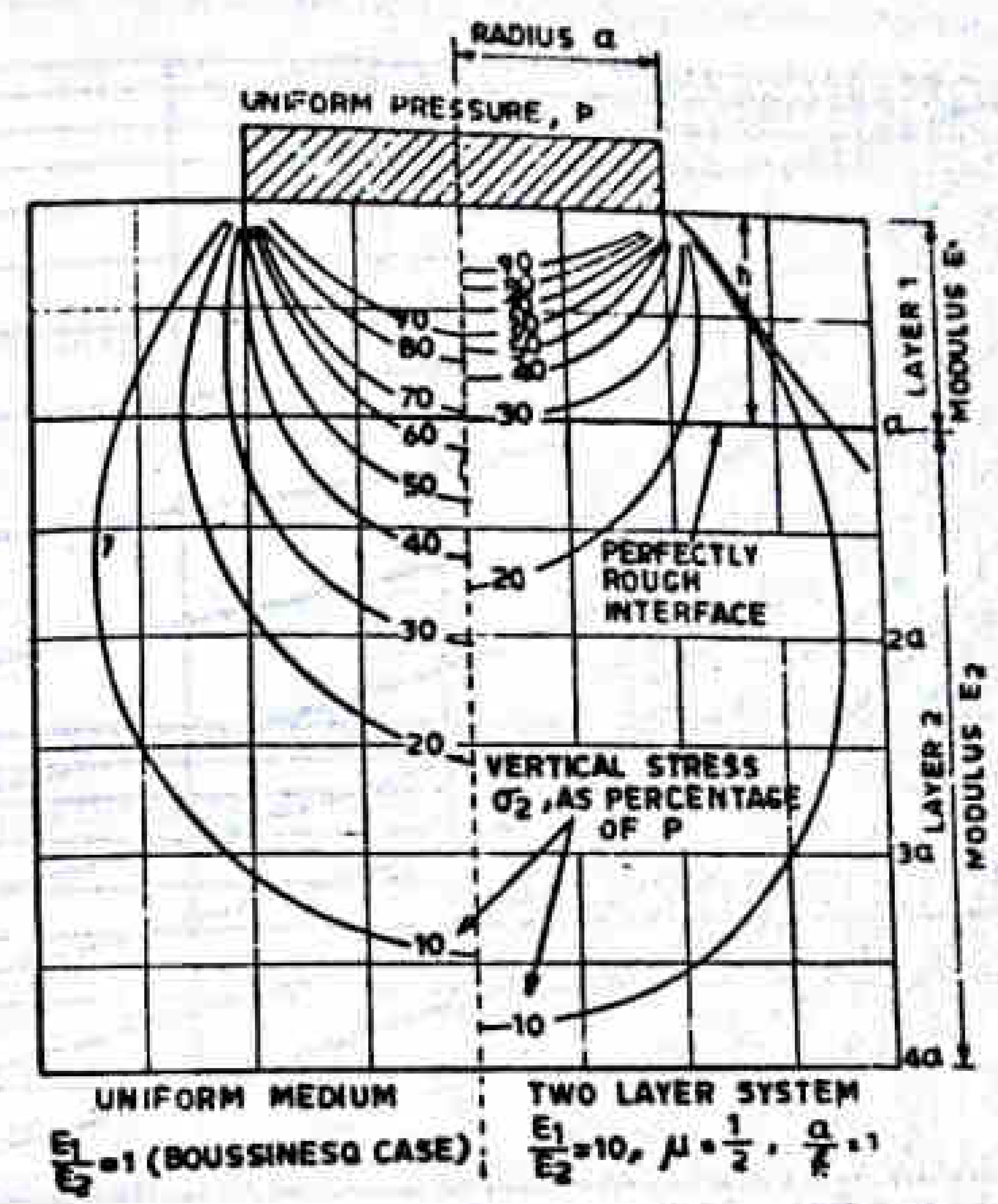


Fig. 7.21 Comparison of Vertical Stress Distribution by Boussinesq and Burmister Approaches

For flexible plate, $\Delta = 1.5\,\dfrac{pa}{E_s}.F_2$

For rigid plate, $\Delta = 1.18\,\dfrac{pa}{E_p}.F_2$

For single layer, h = 0, and $E_s/E_p = 1$ therefore $F_2 = 1$ and these equations reduce to Boussinesq's settlement equation (Eq. 7.2 and 7.3). See Figure 7.22. In the derivations of displacement equations the Poisson's ratio μ is taken as 0.5 both for subgrade and pavement material,

i.e. $\mu_s = \mu_p = 0.5$

The above analysis is adopted by U.S. Navy Department for design of *air field* pavements. It is considered that the layered system analysis can also be applied for design of highway pavements. Following assumptions can be suitably made. The plate diameter for load tests may be taken as 30 cm and design deflection may be taken as 0.5 or 0.25 cm. The design method using Burmister's two-layer theory is illustrated in Example 7.11.

Example 7.11

The plate bearing tests were conducted with 30 cm plate diameter on soil subgrade and over 15 cm base course. The pressure yielded at 0.5 cm deflection are 1.25 kg/cm² and 4.0 kg/cm², respectively.

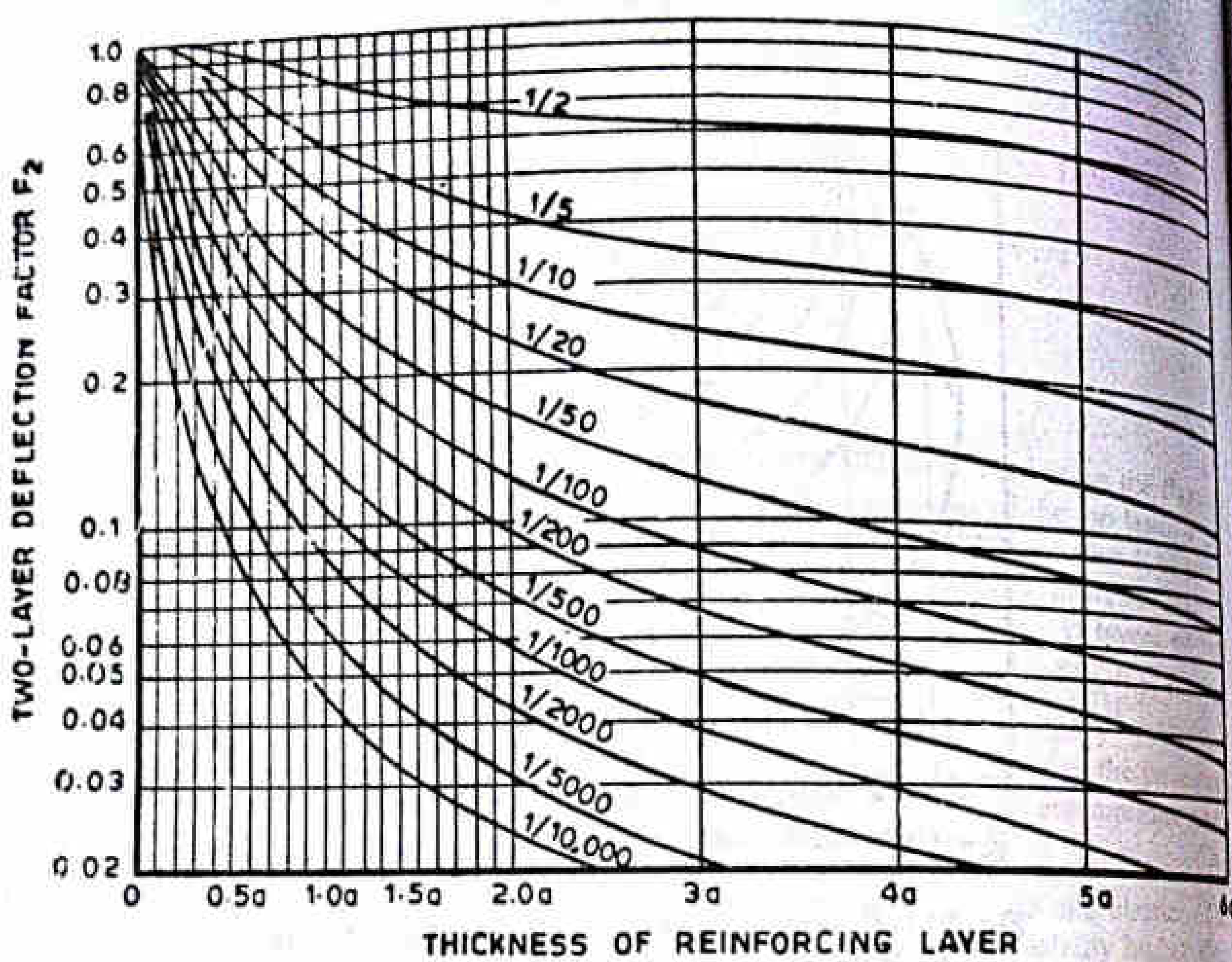


Fig. 7.22 Relationship of F2 and h in a Two-lane System (Burmister's Method)

Design the pavement section for 4100 kg wheel load with tyre pressure of 5 kg/cm^2 for an allowable deflection of 0.5 cm using Burmister's approach.

Solution

(i) Calculate elastic modulus, E_s for soil subgrade or single layer and rigid circular plate,

$$\Delta = 1.18\,\frac{pa}{E_s}\,F_2;\ \text{i.e. } 0.5 = \frac{1.18\times1.25\times15\times1}{E_s}$$

$$F = 1 \text{ (for single layer)}$$

$$E_s = 1.18\,\frac{1.25\times15}{0.5} = 44.2 \text{ kg/cm}^2$$

(ii) Calculate elastic modulus ratio of subgrade to pavement E_s/E_p

$$\Delta = \frac{1.18\,pa}{E_s}\times F_2;\ \text{i.e. } 0.5 = \frac{1.18\times4\times15}{44.2}\times F_2$$

or,

$$F = \frac{0.5\times44.2}{1.18\times4\times15} = 0.312$$

Use Fig. 7.22 and read value of E_s/E_p against $F_2 = 0.312$ and $(h/a) = (15/15) = 1.0$

$$\frac{E_s}{E_p} = \frac{1}{40} \text{ (by interpolation between 1/20 and 1/50)}$$

(iii) Design of flexible pavement for load P = 4100 kg and tyre pressure of 5 kg/cm^2

$$p = 5 \text{ kg/cm}^2$$

$$a = \sqrt{\frac{P}{\pi p}} = \sqrt{\frac{4100}{\pi\times5}} = 16.1 \text{ cm}$$

Deflection for flexible plate (wheel load) is given by :

$$\Delta = \frac{1.5\,pa}{E_s}\times F_2;\ \text{i.e. } 0.5 = \frac{1.5\times5\times16.1\times F_2}{44.2}$$

$$F_2 = \frac{0.5\times44.2}{1.5\times5\times16.1} = 0.183$$

For $F_2 = 0.183$ and $(E_s/E_p) = (1/40)$ using Fig. 7.20, h/a = 2.1

Therefore pavement thickness 'h' is given by :

$$h = 2.1\,a = 2.1\times16 = 33.6 \text{ cm}$$

For airfield pavement design by U. S. Navy method modifications of theoretically calculated thickness have been suggested. It has been recommended that nine trial sections be constructed three each on fill, cut and level areas. In each typical locality, three pavement thickness values equal to 2/3 h, h and 1.5 h are adopted and the actual pavement thickness required for the critical deflection is found. Similar approach is possible in highway pavement design also after calculating the thickness requirement by the elastic layer theory.

7.4 DESIGN OF RIGID PAVEMENTS

7.4.1 General Design Considerations

Cement concrete pavements represent the group of rigid pavements. Here the load carrying capacity is mainly due to the rigidity and high modulus of elasticity of the slab itself i.e., *slab action. H. M. Westergaard* is considered the pioneer in providing the rational treatment to the problem of rigid pavement analysis.

Westergaard considered the rigid pavement slab as a thin elastic plate resting on soil subgrade, which is assumed as a dense liquid. Here it is assumed that the upward reaction is proportional to the deflection, i.e., p = K Δ, where the constant K is defined as modulus of subgrade reaction. The unit of K is kg/cm^2 per cm deflection i.e., kg/cm^3.

Westergaard's Modulus of Subgrade Reaction

The modulus of subgrade reaction, K is proportional to the displacement. The displacement level Δ is taken as 0.125 cm in calculating K as explained in Art. 6.1.8. If p is the pressure sustained in kg/cm^2 by the rigid plate of diameter 75 cm at a deflection Δ = 0.125 cm, the modulus of subgrade reaction K is given by :

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

Relative Stiffness of Slab to Subgrade

A certain degree of resistance to slab deflection is offered by the subgrade. This is dependent upon the stiffness or pressure-deformation properties of the subgrade material. The tendency of the slab to deflect is dependent upon its properties of flexural strength.

The resultant deflection of the slab which is also the deformation of subgrade is a direct measure of the magnitude of subgrade pressure. The pressure deformation characteristics of rigid pavement is thus a function of relative stiffness of slab to that of subgrade.

Westergaard defined this term as the *Radius of relative stiffness*

$$l = \left[\frac{Eh^3}{12K(1-\mu^2)}\right]^{\frac{1}{4}} \qquad (7.15)$$

Here l = radius of relative stiffness, cm

E = modulus of elasticity of cement concrete kg/cm^2

μ = Poisson's ratio for concrete = 0.15

h = slab thickness, cm

K = subgrade modulus or modulus of subgrade reaction, kg/cm^3

Example 7.12

Compute the radius of relative stiffness of 15 cm thick cement concrete slab from the following data :

Modulus of elasticity of cement concrete = 2,10,000 kg/cm^2

Poisson's ratio for concrete = 0.13

Modulus of subgrade reaction, K = (i) 3.0 kg/cm^3 (ii) 7.5 kg/cm^3

Solution

(i) For K = 3.0

$$l = \left[\frac{Eh^3}{12K(1-\mu^2)}\right]^{\frac{1}{4}} = \left[\frac{210000\times15^3}{12\times3(1-0.15^2)}\right]^{\frac{1}{4}} = 67.0 \text{ cm}$$

(i) For K = 7.5

$$t = \left[\frac{210000\times15^3}{12\times7.5(1-0.15^2)}\right]^{\frac{1}{4}} = 53.3 \text{ cm}$$

This indicates that the influence of modulus of subgrade reaction on the slab is relatively small.

The stresses acting on a rigid pavement are ;

(i) wheel load stresses and

(ii) temperature stresses.

Critical Load Position

Since the pavement slab has finite length and width, either the character or intensity of maximum stress induced by the application of a given traffic load is dependent on the location of the load on the pavement surface.

There are three typical locations namely the interior, edge and corner, where differing conditions of slab continuity exist. These are termed as *critical load positions.*

Interior Loading : When load is applied in the interior of the slab surface at any place remote from all the edges.

Edge Loading : When load is applied on an edge of the slab at any place remote from a corner.

Corner Loading : When the centre of load application is located on the bisector of the corner angle formed by two intersecting edges of the slab, and the loaded area is at the corner touching the two corner edges.

Equivalent Radius of Resisting Section

Considering the case of interior loading, the maximum bending moment occurs at the loaded area and acts radially in all directions. With the load concentrated on a small area of the pavement, the question arises as to what sectional area of the pavement is effective in resisting the bending moment. According to Westergaard, the equivalent radius of resisting section is approximated, in terms of radius of load distribution and slab thickness,

$$b = \sqrt{1.6a^2 + h^2} - 0.675\,h \qquad (7.16)$$

Here, b = equivalent radius of resisting section, cm when a is less than 1.724 h

a = radius of wheel load distribution, cm

h = slab thickness, cm

when a is greater than 1.724 h, the value of b = a

Example 7.13

Compute the equivalent radius of resisting section of 20 cm slab, given that the radius of contact area wheel load is 15 cm.

Solution

$$h = 20 \text{ cm}, a = 15$$

$$\frac{a}{h} = \frac{15}{20} = 0.75, < 1.724$$

Therefore

$$b = \sqrt{1.6a^2 + h^2} - 0.675\,h$$

$$= \sqrt{1.6\times15^2 + 20^2} - 0.675\times20 = 14.07 \text{ cm}$$

Maximum stress produced by a wheel load at corner does not exist around the load, but it occurs at some distance X along the corner bisector. Thus is given by the relation : (7.17)

$$X = 2.58\sqrt{al}$$

Here, X = distance from apex of slab corner to section of maximum stress along the corner bisector, cm

a = radius of wheel load distribution, cm

l = radius of relative stiffness, cm

7.4.2 Wheel Load Stresses

A. T. Goldbeck indicated that many concrete slabs failed at the corners. He derived a corner load formula due to a point load at the corner of the slab. Goldbeck's formula for stress due to corner load is given by :

$$S_c = \frac{3P}{h^2} \tag{7.18}$$

Here, S_c = stress due to corner load, kg/cm^2

P = corner load assumed as a concentrated point load, kg

h = thickness of slab, cm

However the assumptions of unsupported corner and concentrated point load at corner have been later found to be severe resulting in very high thickness requirement of slabs.

Westergaard's stress equation for wheel loads

The cement concrete slab is assumed to be a homogeneous, thin elastic plate with subgrade reaction being vertical and proportional to the deflection.

The commonly used equations for theoretical computation of wheel load stresses have been given by Westergaard. He considered three typical regions of the cement concrete pavement slab for the analysis of stresses, as the interior, edge and the corner regions. The critical stresses S_i, S_e and S_c at the typical locations i.e. interior, edge and corner are given in Eq. 7.19 to 7.21.

Interior Loading

$$S_i = \frac{0.316\,P}{h^2}\,[4 \log_{10}(l/b) + 1.069] \tag{7.19}$$

Edge Loading

$$S_e = \frac{0.572\,P}{h^2}\,[4 \log_{10}(l/b) + 0.359] \tag{7.20}$$

Corner Loading

$$S_c = \frac{3\,P}{h^2}\left[1 - \left(\frac{a\sqrt{2}}{l}\right)\right] \tag{7.21}$$

Here,

S_i, S_e, S_c = maximum stress at interior, edge and corner loading, respectively, kg/cm^2

h = slab thickness, cm

P = wheel load, kg

a = radius of wheel load distribution, cm

l = radius of relative stiffness, cm (see Eq. 7.15)

b = radius of resisting section, cm (see Eq. 7.16)

Charts for stress computations

If the slab thickness h is to found for the allowable values of maximum stresses S_i, S_e and S_c trials are required for assumed values of h. Bradbury suggested a simplified procedure by expressing all equations in the general form

$$S = \frac{P}{h^2}\,Q \tag{7.22}$$

He presented charts to find the values of *stress coefficients* Q, from the values of $\left(\frac{l}{b}\right)$ or $\left(\frac{a}{l}\right)$

Khanna et al have given a set of design charts in metric units, in Journal of I.R.C. Volume XXXIII-2 1970 based on Westergaard's equations for computation of wheel load stresses. These charts may be used to find the load stresses at interior edge and corner regions of the cement concrete slab, instead of using Westergaard's stress equations. There is a considerable saving in time in the computation of load stresses by the use of these charts.

Evaluation of wheel load stresses for design

Westergaard's wheel load stress equations for interior, edge and corner have been modified by various investigators based on their research work on cement concrete pavement slabs. The stresses at the edge and corner regions are generally found to be more critical for the design of rigid pavement for highways. The Indian Roads Congress recommends the following two formulas for the analysis of load stresses at the edge and corner regions and for the design of rigid pavements;

(i) Westergaard's edge load stress formula, modified by Teller and Sutherland for finding the load stress S_e in the critical edge region,

$$S_e = 0.529\,\frac{P}{h^2}\,(1 + 0.54\,\mu) \times (4 \log_{10} l/b + \log_{10} b - 0.4048) \tag{7.23}$$

(ii) Westergaard's corner load stress analysis modified by Kelley for finding the load stress S_c at the critical corner region,

$$S_c = \frac{3\,P}{h^2}\left[1 - \left(\frac{a\sqrt{2}}{l}\right)^{1.2}\right] \tag{7.24}$$

where, S_e = load stress at the edge region, kg/cm^2

S_c = load stress at the corner region, kg/cm^2

P = design wheel load, kg

h = thickness of CC pavement slab, cm

μ = Poisson's ratio of the CC slab

E = modulus of elasticity of the CC, kg/cm^2

K = reaction modulus of pavement foundations (i.e., base course, sub-base course or subgrade), kg/cm^3

l = radius of relative stiffness, cm = $\left[\dfrac{E h^3}{12 K (1-\mu^2)}\right]^{\frac{1}{4}}$

b = radius of equivalent distribution of pressure, cm,

b = a, when $\frac{a}{h} \geq 1.724$ and

b = $\sqrt{1.6 a^2 + h^2} - 0.675$ b, when $\frac{a}{h} \leq 1.724$

a = radius of load contact, cm (assumed circular in shape)

The above equations 7.23 and 7.24 for finding load stresses at the edge and corner regions are presented in the form of stress charts by the IRC and these are shown in Fig. 7.23 and 7.24. These charts are applicable for a particular set of design parameters only viz. : P = 5100 kg, a = 15 cm, E = 3 × 10^5 kg/cm^2, μ = 0.15; but different curves are given for different values of K between 6.0 and 30 kg/cm^3. The design curves are for slab thickness values, h = 14 to 25 cm. These stress charts are very handy and save considerable time when the stresses are to be evaluated for various trial thickness of the slab while designing a pavement.

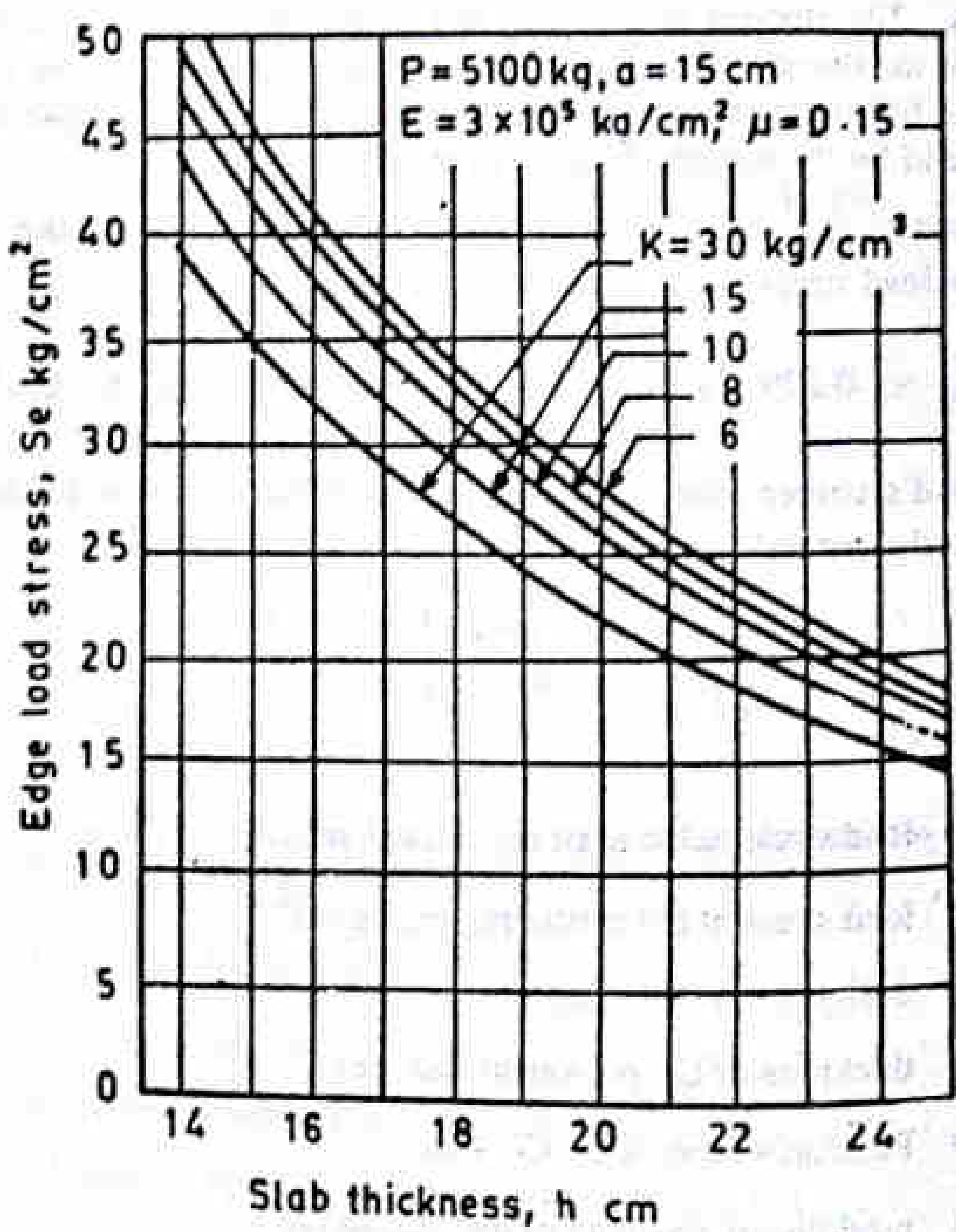


Fig. 7.23 Edge Load Stress Chart (IRC)

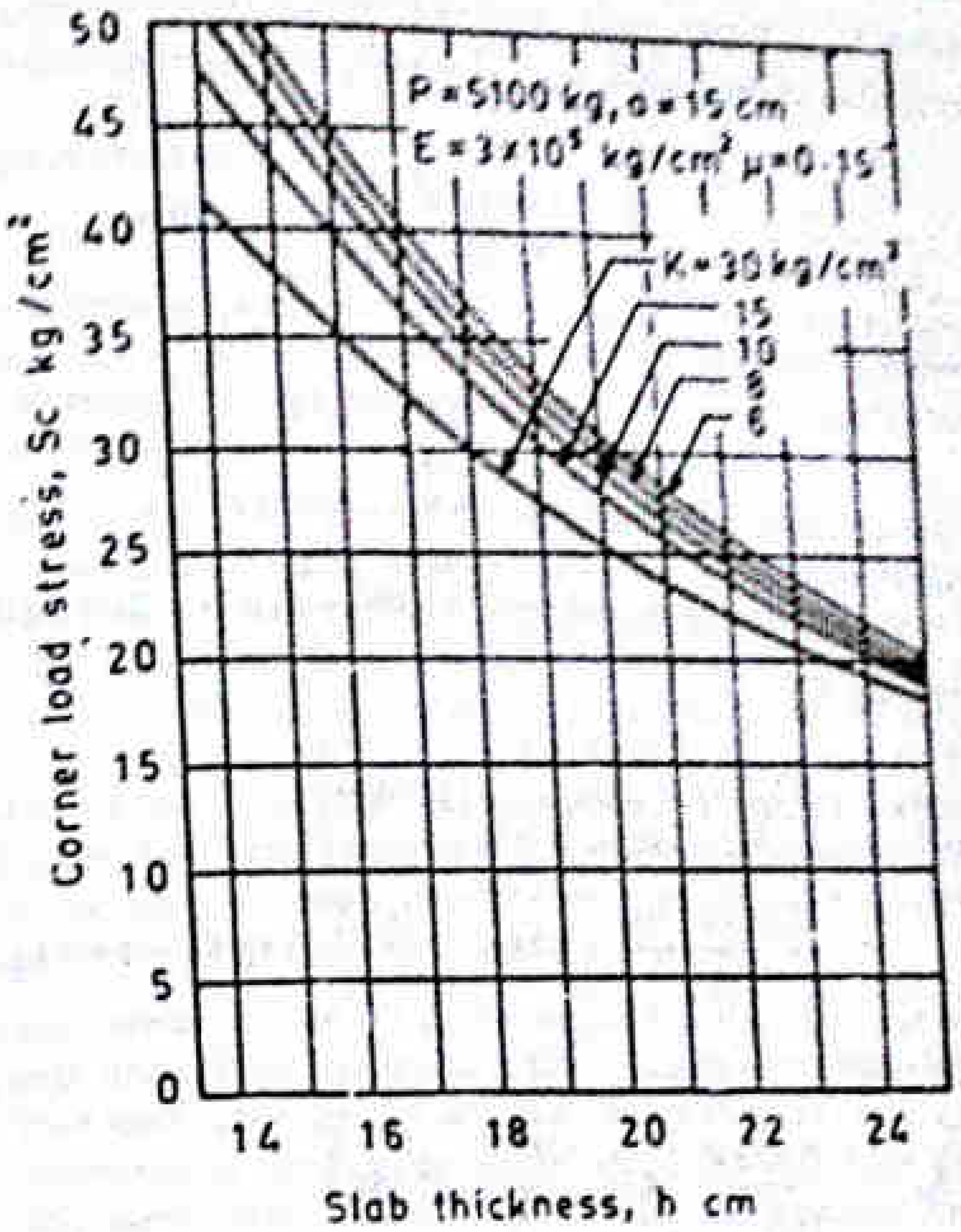


Fig. 7.24 Corner Load Stress Chart (IRC)

Load Stress Parameters

Wheel load P = 5100 kg.

Radius a = 15 cm

Elastic modulus of cement concrete E = 3 × 10^5 kg/cm^2.

Poisson's ratio μ = 0.15

Example 7.15

Calculate the stresses at interior, edge and corner regions of a cement concrete pavement using Westergaard's stress equations. Use the following data :

Wheel load, P = 5100 kg

Modulus of elasticity of cement concrete, E = 3.0 × 10^5 kg/cm^2

Pavement thickness, h = 18 cm

Poisson's ratio of concrete, μ = 0.15

Modulus of subgrade reaction, K = 6.0 kg/cm^3

Radius of contact area, a = 15 cm

Solution

Radius of relative stiffness (l) is given by

$$l = \left[\frac{E\,h^3}{12\,K\,(1-\mu^2)}\right]^{\frac{1}{4}} = \left[\frac{3.0\times10^5\times18^3}{12\times6\,(1-0.15^2)}\right]^{\frac{1}{4}} = 70.6 \text{ cm}$$

The equivalent of resisting section is given by

$$a/h = 15/8 = 0.833 < 1.74$$

$$b = \sqrt{1.6a^2+h^2} - 0.675\,h$$

$$= \sqrt{1.6\times15^2+18^2} - 0.675\times18 = 14.0 \text{ cm}$$

Stress at the interior, (S_i)

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

$$= \frac{0.316\times5100}{18^2}\left[4\log_{10}\left(\frac{70.6}{14.0}\right)+1.069\right] = 19.3 \text{ kg/cm}^2$$

Stress at the Edge (S_e)

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

$$= \frac{0.572\times5100}{18^2}\,[4\times0.7027+0.359] = 28.54 \text{ kg/cm}^2$$

Stress at the Corner (S_c)

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

$$= \frac{3\times5100}{18^2}\left[1-\left(\frac{15\sqrt{2}}{70.6}\right)^{0.6}\right] = 24.27 \text{ kg/cm}^2$$

Example 7.16

A CC pavement of thickness 20 cm rests over a WBM base course with modulus of reaction 30 kg/cm^3. Find the load stresses at the edge and corner regions under a wheel load of 5100 kg unit IRC stress charts. (Assume a = 15 cm, E = 3 × 10^5 kg/cm^2 and μ = 0.15).

Solution

Refer edge load stress chart (Fig. 7.23). Using the curve for K = 30 kg/cm^3 corresponding to h = 20 cm, edge load stress S_e = 22.0 kg/cm^2.

Referring to corner load stress chart (Fig. 7.24), using curve K = 30 and corresponding to h = 20, corner load stress S_c = 25.5 kg/cm^2.

7.4.3 Temperature Stresses

Westergaard's Concept for Temperature Stresses

Temperature stresses are developed in cement concrete pavement due to variation in slab temperature. The variation in temperature across the depth of the slab is caused by *daily variation* whereas an overall increase or decrease in slab temperature is caused by *seasonal variation* in temperature.

During the day, the top of the pavement slab gets heated under the sun light when the bottom of the slab still remains relatively colder. The maximum difference in temperature between the top and bottom of the pavement slab may occur at some period after the mid-noon. This causes the slab to warp or bend, as the warping is resisted by the self weight of the slab, *warping stresses* are developed late in the evening, the bottom of the slab gets heated up due to heat transfer from the top and as the atmospheric temperature falls, the top of the slab becomes colder resulting in warping of the slab in the opposite direction and there is a reversal in warping stresses at the different regions of the slab. Thus the daily variation in temperature causes warping stresses in reverse directions at the corner, edge and interior regions of the slab.

During summer season as the mean temperature of the slab increases, the concrete pavement expands towards the expansion joints. Due to the frictional resistance at the interface (which depends upon the self weight of the slab and the coefficient of friction at the interface), compressive stress is developed at the bottom of the slab as it tends to expand. Similarly during winter season, the slab contracts causing tensile stress at the bottom due to the frictional resistance again opposing the movement of the slab. Thus *frictional stresses* are developed due to seasonal variation in temperature. The frictional stress will be zero at the free ends and at expansion joints and increases upto a maximum value towards the interior and there-after remains constant.

Temperature thus tends to produce two types of stresses in a concrete pavement. These are

(i) warping stresses and

(ii) frictional stresses.

Warping stresses

Whenever the top and bottom surfaces of a concrete pavement simultaneously possess different temperatures, the slab tends to warp downward or upward inducing warping stresses. See Fig. 8.18, under Highway Construction chapter.

The difference in temperature between the top and bottom of the slab depends mainly on the slab thickness and the climatic conditions of the region.

By the time the top temperature increases to t_1 degrees, the bottom temperature may be only t_2 degrees and the difference between the top and bottom of the slab would be $(t_1 - t_2)$ = t degrees.

Assuming straight line variation in temperature across the pavement depth, the temperature at mid depth or average temperature of slab would be $(t_1 + t_2)/2$.

If the slab has no restraint then the unit elongation of the top fibres and also the contraction of the bottom fibre due to relative temperature condition, each would be equal to Eet/2 where e is the thermal coefficient of concrete. Westergaard worked out the stresses due to the warping of concrete slabs.

Now introducing the effect of Poisson's ratio the stresses at the interior, region in longitudinal and transverse directions as given by *Bradbury* are expressed by the following equations :

$$St_{(i)} = \frac{Eet}{2}\left[\frac{C_x + \mu C_y}{1-\mu^2}\right] \quad (7.25)$$

Here, $St_{(i)}$ = warping stress at interior, kg/cm^2

E = modulus of elasticity of concrete, kg/cm^2

e = thermal coefficient of concrete per °C

t = temperature difference between the top and bottom of the slab in degree C

C_x = coefficient based on L_x/l in desired direction

C_y = coefficient based on L_y/l in right angle to the above direction

μ = Poisson's ratio (may be taken as 0.15)

L_x and L_y are the dimensions of the slab considering along X and Y directions along the length and width of slab.

The values of the warping stress coefficients C_x and C_y for cement concrete pavement are taken from the chart developed by Bradbury. See Fig. 7.25. The warping stress at the edge region is given by :

$$St_{(e)} = \frac{C_x\, E.e.t}{2} \quad (7.26a)$$

or

$$= \frac{C_y\, E.e.t}{2} \text{ (whichever is higher)} \quad (7.26b)$$

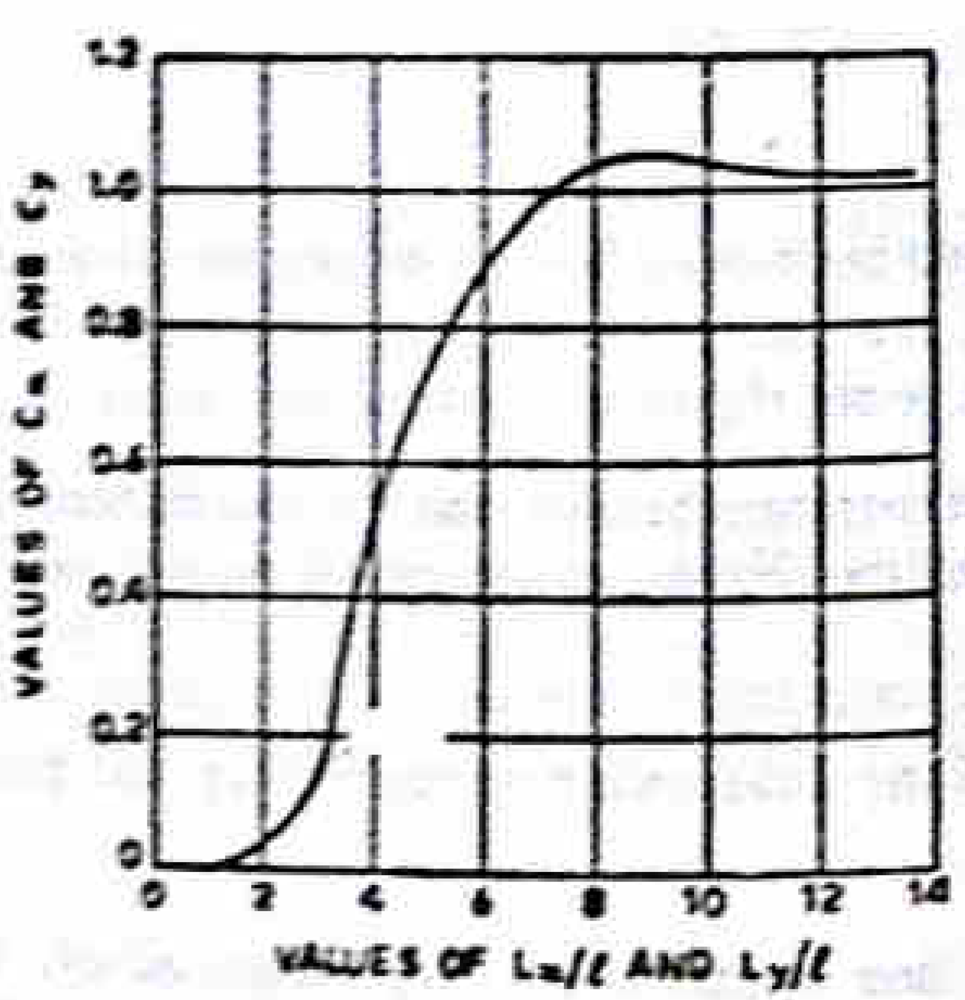


Fig. 7.25 Warping Stress Coefficient

For corner region, warping stress is given by :

$$St_{(e)} = \frac{E.e.t}{3(1-\mu)}\sqrt{\frac{a}{l}} \quad (7.27)$$

Here, a is radius of contact and l is the radius of relative stiffness.

Example 7.17

Determine the warping stresses at interior, edge and corner regions in a 25 cm thick concrete pavement with transverse joints at 11 n interval and longitudinal joints at 3.6 m intervals. The modulus of subgrade reaction (K) is 6.9 kg/cm^3. Assume temperature differential for day conditions to be 0.6°C per cm slab thickness. Assume radius of loaded area as 15 cm for computing warping stress at the corner. Additional data are given below :

$$e = 10 \times 10^{-6} \text{ per °C}$$

$$E = 3 \times 10^5 \text{ kg/cm}^2$$

$$\mu = 0.15$$

Solution

Calculate the radius of relative stiffness

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

$$= \left[\frac{3\times10^5 \times 25^3}{12\times6.9(1-0.15^2)}\right]^{0.25} = 87.2 \text{ cm}$$

$$\frac{L_x}{l} = \frac{1100}{87.2} = 12.61$$

From Fig. 7.25, $C_x = 1.03$

Also $\frac{L_y}{l} = \frac{360}{87.2} = 4.13,\ C_y = 0.55$

$$t = 25 \times 0.6 = 15\ °C$$

Interior warping stress from Equation 7.25

$$St_{(i)} = \frac{3\times10^5 \times 10\times10^{-6} \times 15}{2}\left[\frac{1.03+0.15\times0.55}{1-0.15^2}\right]$$

$$= 25.61 \text{ kg/cm}^2$$

Longitudinal edge stresses due to warping from Eq. 7.26 (Using C_x value, as it is higher than C_y)

$$St_{(e)} = \frac{1.03 \times 3 \times 10^5 \times 10 \times 10^{-6} \times 15}{2} = 23.18 \text{ kg/cm}^2$$

Warping stress at the corner region from Eq. 7.27.

$$St_{(c)} = \frac{3 \times 10^5 \times 10 \times 10^{-6} \times 15}{3(1-0.15)} \sqrt{\frac{15}{87.2}} = 6.36 \text{ kg/cm}^2$$

Thomlinson's Temperature Stress Analysis

J. Thomlinson in 1940 provided an analytical approach for temperature stress computations. From actual measurements of temperature in cement concrete pavements using *Thermocouples*, it has been seen that the temperature gradient across the slab thickness is infact curvilinear as against in the assumption of straight line variation by Westergaard. Thomlinson developed an analysis which yields results more close to the experimental data and lower than Westergaard's equations. Details of this analysis are not discussed here.

Frictional stresses

Due to uniform temperature rise and fall in the cement concrete slab, there is an overall expansion and contraction of the slab. Since the slab is in contact with soil subgrade or the sub-base, the slab movements are restrained due to the friction between the bottom layer of the pavement and the soil layer. This frictional resistance therefore tends to prevent the movements thereby inducing the frictional stress in the bottom fibre of the cement concrete pavement. Stresses in slabs resulting due to this phenomenon vary with slab length. In short, slab stress induced due to this is negligibly small whereas in long slabs, which would undergo movements of more than 0.15 cm, higher amount of frictional stress develops.

Equating, total force developed in the cross section of concrete pavement due to movement and frictional resistance due to subgrade restraint in half the length of the slab,

$$S_f \times h \times B \times 100 = B \times \frac{L}{2} \times \frac{h}{100} \times W \times f$$

$$S_f = \frac{WLf}{2 \times 10^4} \quad (7.28)$$

Here S_f = unit stress developed in cement concrete pavement, kg/cm^2

W = unit weight of concrete, kg/cm^3 (about 2400 kg/m^3)

f = coefficient of subgrade restraint (maximum value is about 1.5)

L = slab length, metre

B = Slab width, metre

7.4.4 Combination of Stresses

It is necessary to consider the conditions under which the various stresses in cement concrete pavements would combine to give the most critical combinations.

The following conditions are considered to provide the critical combinations :

(i) *During summer* : The critical combinations at interior and edge regions during mid day occurs when the slab tends to warp downward. During this period maximum tensile stress is develop at the bottom fibre due to warping and this is cumulative with the tensile stress due to the loading. However the frictional stress is compressive during expansion. The load stress at edge region is higher than the interior.

Critical combination of stresses = load stress + warping stress – frictional stress), *at edge region*.

(ii) *During winter* : The critical combination of stresses at the above regions occurs at the bottom fibre when the slab contracts and the slab warps downward during the mid day. The frictional stress is tensile during contraction.

The critical stress combination = (load stress + warping stress + frictional stress), *at edge region*.

Since the differential temperature t is of lower magnitude during winter than in summer, the combination (i) may be worst for most of the regions in this country.

(iii) At corner region, the critical combination occurs at the top fibre of the slab, when the slab warps upwards during the mid nights. There is no frictional stress at the corner region.

The critical stress combination = (load stress + warping stress), *at corner regions*.

Example 7.18

A CC pavement slab of thickness 20 cm is constructed over a granular sub-base having modulus of reaction 15 kg/cm^2. The maximum temperature difference between the top and bottom of the slab during summer day and night is found to be 18°C. The spacing between the transverse contraction joint is 4.5 m and that between longitudinal joints is 3.5 m. The design wheel load is 5100 kg, radius of contact area is15 cm, E value of CC is 3×10^5 kg/cm^2, Poisson's ratio is 0.15, and coefficient of thermal expansion of CC is 10×10^{-6} per °C and friction coefficient is 1.5. Using the edge and corner load stress charts given by the IRC and the chart for the warping stress coefficient, find the worst combination of stresses at the edge.

Solution

(i) Edge Region

(a) Edge load stress from chart (Fig. 7.23).

for h = 20 cm and K = 15 kg/cm^3, S_e = 24.0 kg/cm^2

(b) Warping stress at edge :

$$\text{Radius of relative stiffness } l = \left[\frac{3 \times 10^5 \times 20^3}{12 \times 15(1-0.15^2)}\right]^{\frac{1}{4}} = 60.8 \text{ cm}$$

Length of slab L_x = 4.5 m = 450 cm

Warping stress coefficient, C_x from Fig. 7.25, at

$$\frac{L_x}{l} = \frac{450}{60.8} = 7.4,\ C_x = 1.02$$

Similarly at $\frac{L_y}{l} = \frac{350}{60.8} = 5.75,\ C_y = 0.87;$

$$t = 18\ ^\circ C$$

Maximum warping stress at edge,

$$S_{te} = \frac{Eet}{2}\ .C_x = \frac{1}{2} \times 3 \times 10^5 \times 10 \times 10^{-6} \times 18 \times 1.02 = 27.54\ kg/cm^2$$

(c) Frictional stress :

From Eq. 7.28, $S_f = \frac{W.L_x.f}{2 \times 10^4} = \frac{2400 \times 4.5 \times 1.5}{2 \times 10^4} = 0.81\ kg/cm^2$

(d) Combined stress at edge region :

Critical combination of stress during summer mid-day = load stress + warping stress – frictional stress

$$= 24.0 + 27.54 - 0.81 = 50.73\ kg/cm^2$$

(ii) Corner Region

(a) Load stress :

From chart 7.24 for h = 20 and K = 15,

$$S_c = 28.0\ kg/cm^2$$

(b) Max. warping stress :

From Eq. 7.27,

$$St_c = \frac{E.e.t}{3(1-\mu)}\sqrt{\frac{a}{l}} = \frac{3 \times 10^5 \times 10 \times 10^{-6} \times 18}{3(1-0.15)}\sqrt{\frac{15}{60.8}} = 9.15\ kg/cm^2$$

(c) Frictional stress : This is zero at corner region

(d) Combined stress at the corner region :

The critical combination of stress in summer mid-night = load stress + warping stress

$$= 28.0 + 9.15 = 37.15\ kg/cm^2$$

(It may be noted that the critical combination of stresses at the edge region is higher than that at the corner under the identical condition of pavement, load and temperature).

7.4.5 Design of Joints in Cement Concrete Pavements

Various types of joints provided in cement concrete pavements to reduce the temperature stresses are expansion joint, contraction joints and warping joints. If expansion and contraction joints are properly designed and constructed, there is no need of providing warping joints, in addition. Expansion joint spacing is designed based on the maximum temperature variations expected and the width of joint. The contraction

joint spacing design is governed by the anticipated frictional resistance and allowable tensile stress in concrete during the initial curing period, or the amount of reinforcement if any. The spacing between the expansion joints is so adjusted that the contraction joints have equal spacing. *Dowel bars* are provided at expansion joints and some times at contraction joints also. The size and spacing of the dowel bars are designed and are also governed by standard specification based on practical considerations. Longitudinal joints in cement concrete pavements are constructed with suitable *tie bars*. The design considerations include diameter, spacing and length of the bars.

Spacing of expansion joint

The width or the gap in expansion joint depends upon the length of slab. Greater the distance between the expansion joints, the greater is the width required of the gap for expansion. The use of wide expansion joint space should be avoided as it would be difficult to keep them properly filled in when the gap widens during winter season. The dowels would develop high bending and bearing stresses with wider openings. It is recommended not to have a gap more than 2.5 cm in any case. The IRC has recommended that the maximum spacing between expansion joints should not exceed 140 m for rough interface layer.

If 'δ' is the maximum expansion in a slab of length L_e with a temperature rise from T_1 to T_2.

δ' = L_e C (T_2 – T_1) where C is the thermal expansion of concrete per degree rise in temperature.

The joint filler may be assumed to be compressed up to 50 percent of its thickness and therefore, the expansion joint gap should be twice the allowable expansion in concrete, i.e., 2δ'. From the relation given above, if δ' is half the joint width, the spacing of expansion joint L_e is given by the equation :

$$L_e = \frac{\delta'}{100\,C\,(T_2 - T_1)} \qquad (7.29)$$

Example 7.19

The width of expansion joint gap is 2.5 cm in a cement concrete pavement. If the laying temperature is 10°C, and the maximum slab temperature in summer is 54°C, calculate the spacing between expansion joints. Assume coefficient of thermal expansion of concrete as 10×10^{-6} per °C.

Solution

$$\delta' = \frac{2.5}{2}$$

$$= 1.25\ cm$$

$$T_2 - T_1 = 54 - 10 = 44°C$$

$$L = \frac{1.25}{100 \times 10 \times 10^{-6} \times 44}$$

$$= 28.5\ m$$

Spacing of contraction joints

The slab contracts due to the fall in slab temperature below the construction temperature. Also during the initial curing period, shrinkage occurs in cement concrete. This movement is resisted by the subgrade drag or friction between the bottom fibre of the slab and the subgrade; see Fig. 7.26. If L_c is the slab length in metre, the maximum stress occurs at half the length.

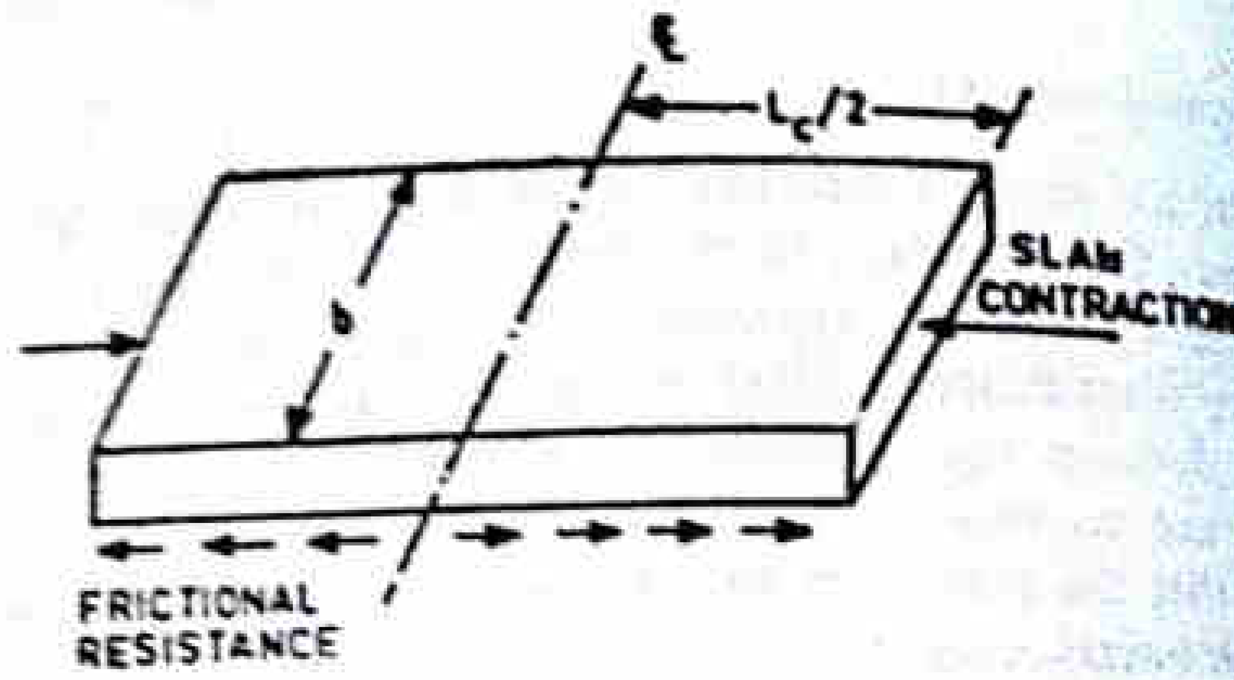


Fig. 7.26 Slab Contraction and Frictional Resistance

Total frictional resistance upto distance $L_c/2 = W \times b \times (L_c/2) \times (h/100) \times f$

Allowable tension in cement concrete $= S_c \times h \times b \times 100$

Equating the above two values,

$$\frac{b\,L_c\,h\,f}{200} = 100\, S_c\, h\, b$$

Length of slab to resist the frictional drag, i.e., spacing of contraction joints,

$$L_c = \frac{2 S_c}{Wf} \times 10^4 \qquad (7.30)$$

Here, L_c = slab length or spacing between contraction joints, m

h = slab thickness, cm

f = coefficient of friction, (maximum value is about 1.5)

W = unit weight of cement concrete, kg/m^3 (2400 kg/m^3)

S_c = allowable stress in tension in cement concrete, kg/cm^2 (0.8 kg/cm^2)

Since the contraction or shrinkage cracks develop mainly during initial period of curing, a very low value of S_c is considered in design. The permissible stress is generally kept as low as about 0.8 kg/cm^2.

Spacing of Contraction Joints when Reinforcement is provided

If it is assumed that the reinforcement takes the entire tensile force in the slab, caused by the frictional resistance of subgrade and *hair cracks* are allowed, then

$$W \times b \times \frac{L_c}{2} \times \frac{h}{100} \times f = S_s A_s$$

$$L_c = \frac{200\, S_s\, A_s}{b\, h\, W\, f} \qquad (7.31)$$

Here, A_s = total area of steel, cm^2 across the slab width

L_c = spacing between contraction joints, m

b = slab width, m

h = slab thickness, cm

W = unit weight of cement concrete, kg/m^3 (2400)

f = coefficient of friction (1.5 max)

S_s = allowable tensile stress in steel, kg/cm^2 (1400)

Example 7.20

Determine the spacing between contraction joints for 3.5 meter slab width having thickness of 20 cm and f = 1.5, for the following two cases :

(i) for plain cement concrete, allowable S_c = 0.8 kg/cm^2

(ii) for reinforcement cement concrete, 1.0 cm dia. Bars at 0.30 m spacing

Solution

Case (i) For Plain Cement Concrete Slab (without reinforcement)

Assume unit weight of CC, W = 2400 kg/m^3

Using Eq. 7.30 spacing between contraction joints,

$$L_c = \frac{2 S_c}{Wf} \times 10^4 = \frac{2 \times 0.8 \times 10^4}{2400 \times 1.5} = 4.44 \text{ m}$$

Case (ii) For Reinforced Cement Concrete Slab

Total cross sectional area of steel, A_s in one direction along the slab width

$$A_s = \frac{3.5 \times \pi \times 1.0^2}{0.3 \times 4} = 9.16 \text{ cm}^2$$

Using Eq. 7.29, spacing between contraction joints,

$$L_c = \frac{200\, S_s\, A_s}{b\, h\, W\, f} = \frac{200 \times 1200 \times 9.16}{3.5 \times 20 \times 2400 \times 1.5} = 8.72 \text{ m}$$

Example 7.21

The maximum increase in temperature is expected to be 26°C after the construction of a CC pavement. If the expansion joint gap is 2.2 cm, design the spacings between the expansion and contraction joints. Assume plain cement concrete construction, with thermal coefficient = 10 × 10^{-6} per°C, unit weight = 2400 kg/m^3, allowable stress in tension during initial period of curing = 0.8 kg/cm^2 and the coefficient of friction of the interface = 1.4.

Solution

Spacing between contraction joints in plain CC pavement,

$$L_c = \frac{2S_c}{Wf} \times 10^4 = \frac{2 \times 0.8 \times 10^4}{2400 \times 1.4} = 4.76 \text{ m}$$

Maximum spacing suggested by the IRC is 4.5 m for plain CC pavements and so adopt $L_c = 4.5$ m

Spacing between expansion joints,

$$L_e = \frac{\delta'}{100\,C\,(T_2 - T_1)}$$

$$= \frac{1.1}{100 \times 10 \times 10^{-6} \times 26} = 42.3 \text{ m}$$

Therefore provide spacing of expansion joints = 9 × 4.5 = 40.5 m

(As 10 × 4.5 = 45.0 m which is higher than 42.3 m, expansion joints are provided after eight contraction joints or after the ninth slab).

Design of Dowel Bar

Dowel bars of expansion joints are mild steel round bars of short length. Half length of this bar is bonded in one cement concrete slab and the remaining portion is embedded in adjacent slab, but is kept free for the movement during expansion and contraction of the slab. The dowel bars allow opening and closing of the joint, maintaining the slab edges at the same level, and the load transference is effected from one slab to the other. This is explained below :

If dowel bars are not provided at the transverse joint (as in Fig. 7.27-a) the loaded slab would deflect by say, dx_1 under load P. The adjacent slab across the expansion joint does not participate in load bearing and it does not deflect at all.

When the joint is provided with dowel bar system and the load P is applied on the first slab under the same conditions, the loaded slab deflects by the magnitude say dx_2. (See Fig. 7.27-b) and the adjacent slab also undergoes a deflection, dx_3 due to the dowel bar transferring part of the load. Theoretically, if the dowel bars are rigidly embedded in the concrete slabs on both sides, dx_2 should be equal to dx_3. It has however been observed that $dx_3 < dx_2$ and their ratio depends upon the thickness of the slab, size and diameter of dowels and their spacing.

It may be noted that $dx_1 > dx_2$ or dx_3. Accordingly, the stress caused in loaded slab is greater when there are no dowels than the slab joint with dowel bars. This is logical as the stress magnitudes are related with deflections. The pressure distribution along the dowel bar under the load on one slab is illustrated in Fig. 7.27 c as per *Friberg's* analysis. Points of pressure reversal a, would determine the criterion for determining the length of dowel bars.

In the design of dowels, the load transference is worked out considering the capacity of the dowel system. The capacity depends upon variable like, pavement thickness, subgrade modulus, the relative stiffness and spacing and size of dowels.

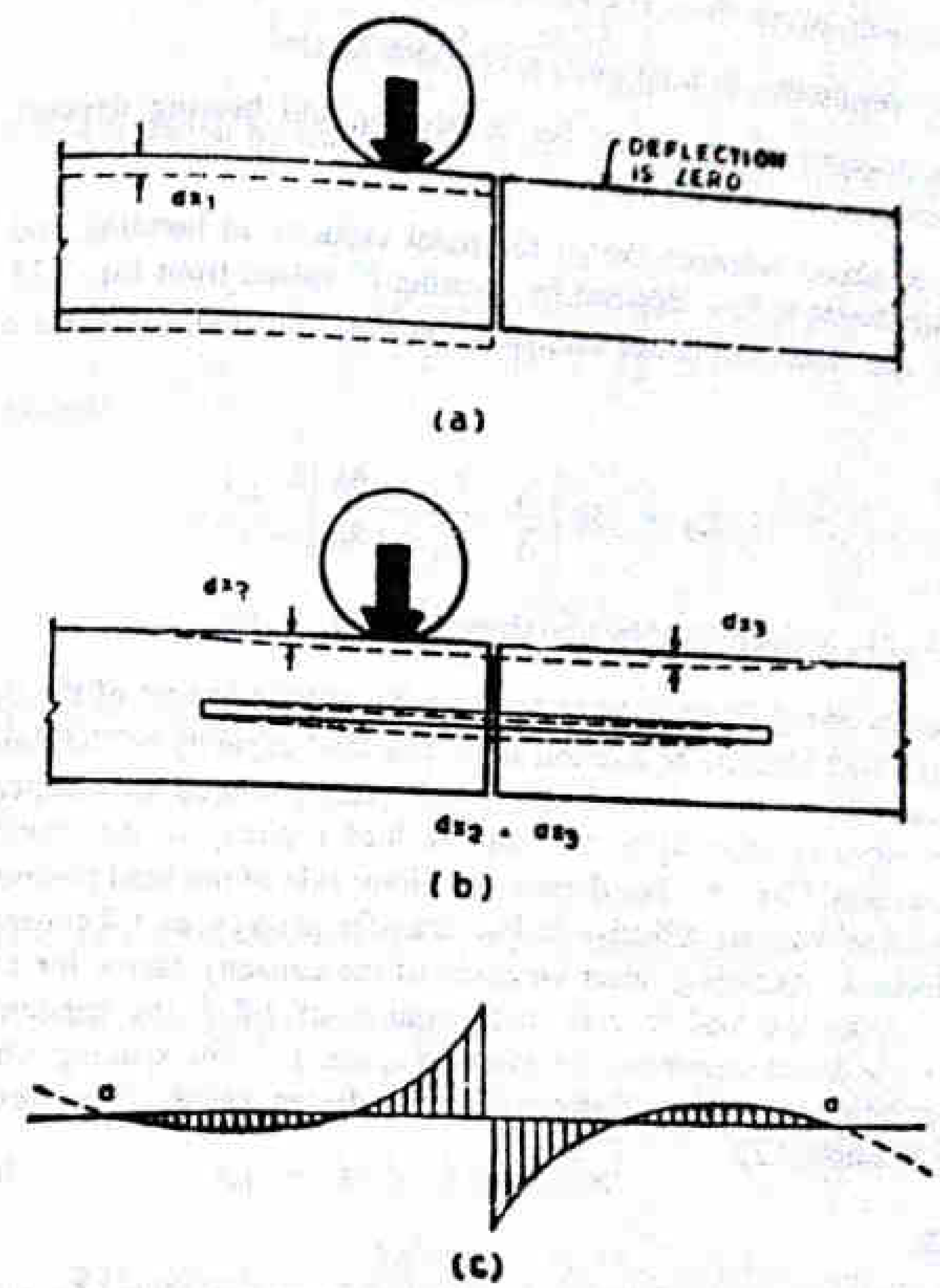


Fig. 7.27 Functioning of Dowel Bar

The IRC recommends that dowel bar system may be designed on the basis of Bradbury's analysis for load transfer capacity of a single dowel bar in shear, bending and bearing in concrete. These values are given below :

For shear in the bar, $P' = 0.785\, d^2 F_s$ (7.32)

For bending in the bar, $P' = \dfrac{2 d^3 F_f}{L_d + 8.8\,\delta}$ (7.33)

For bearing on concrete, $P' = \dfrac{F_b L_d^2 . d}{12.5\,(L_d + 1.5\,\delta)}$ (7.34)

where, P' = load transfer capacity of a single dowel bar, kg

d = diameter of dowel bar, cm

L_d = total length of embedment of dowel bar, cm

δ = joint width, cm

F_s = permissible shear stress in dowel bar, kg/cm²

F_f = permissible flexural stress in dowel bar kg/cm^2

F_b = permissible bearing stress in concrete, kg/cm^2

The load capacity of the dowel bar in bending and bearing depend on the total embedded length L_d on both the slabs.

In order to obtain balanced design for equal capacity in bending and bearing, the length of embedment is first obtained by equating P' values from Eq. 7.33 and 7.34 for the assumed joint width and dowel diameter. On simplification, the value of L_d is given by

$$L_d = 5d\left[\frac{F_f}{F_b}\times\frac{L_d+1.5\delta}{L_d+8.8\delta}\right]^{\frac{1}{2}} \qquad (7.35)$$

The value of L_d is determined by trial from Eq. 7.35.

The minimum dowel length is taken as (L_d + δ), and the lowest of the three values of P' taken as the load capacity of a dowel bar. The load capacity of the dowel system or group is assumed to be 40% of the design wheel load. The required load capacity factor of the dowei group is obtained by dividing the load capacity of the group by the load capacity of one dowel bar, P'. The distance on either side of the load position upto which the group of dowel bars are effective in load transfer is taken as 1.8 times the radius of relative stiffness, *l*. Assuming linear variation of the capacity factor for a single dowel bar from 1.0 under the load to zero at a distance of 1.8 *l*, the capacity factors are calculated for the dowel system for the assumed spacings. The spacing which conforms to the required capacity factor is selected as the design value. The design steps are illustrated in Example 7.22.

Example 7.22

Design the size and spacing of dowel bars at the expansion joints of a cement concrete pavement of thickness 25 cm with radius of relative stiffness 80 cm, for a design wheel load of 5000 kg. Assume load capacity of the dowel system as 40% of the design wheel load. Joint width is 2.0 cm, permissible shear and flexural stresses in dowel bar are 1000 and 1400 kg/cm^2 respectively and permissible bearing stress in CC is 100 kg/cm^2.

Solution

Length of Dowel Bar, L

Assume the diameter of the dowel bar, d = 2.5 cm

P = 5000 kg
l = 80 cm
F_s = 1000 kg/cm^2
F_b = 100 kg/cm^2
H = 25 cm
δ = 2 cm
F_f = 1400 kg/cm^2

For equal capacity of dowel bar in bending and bearing (Eq. 7.35),

$$L_d = 5\times 2.5\left[\frac{1400}{100}\times\frac{L_d+1.5\times 2}{L_d+8.8\times 2}\right]^{\frac{1}{2}}$$

$$= 12.5\left[14\times\frac{L_d+3}{L_d+17.6}\right]^{\frac{1}{2}}$$

Solution of this equation by trial method is simple. Therefore as a first trial assume L_d = 45 cm.

$$L_d = 12.5\left[14\times\frac{45+3}{45+17.6}\right]^{\frac{1}{2}} = 40.95$$

Which is less than 45

Assume L_d = 40.5

$$L_d = 12.5\left[14\times\frac{40.5+3}{40.5+17.6}\right]^{\frac{1}{2}} = 40.47$$

Therefore total length of embedment, L_d = 40.5 cm

Minimum length of dowel bar required, L = L_d + δ
= 40.5 + 2.0 = 42.5 cm

Therefore provide 2.5 cm diameter dowel bars of length 45 cm.

Load Transfer Capacity of Single Dowel Bar, P'

$$P'(\text{shear}) = 0.785\,d^2 F_s = 0.785\times 2.5^2\times 1000 = 4906 \text{ kg}$$

Actual value of L_d = 45.0 − 2.0 = 43 cm

$$P'(\text{bending}) = \frac{2d^3 F_f}{L_d+8.8\,\delta} = \frac{(2\times 2.5^3\times 1400)}{(43+8.8\times 2)} = 722 \text{ kg}$$

$$P'(\text{bearing}) = \frac{F_b\,L_d^{\,2}.d}{12.5(L_d+1.5\,\delta)} = \frac{100\times 43^2\times 2.5}{12.5(43+3)} = 804 \text{ kg}$$

Taking the lowest of the three values for design, load capacity of a dowel bar. P' (design) = 722kg.

Required Load Capacity Factor

Load capacity of the dowel group = 40% of P = 0.4 × 5000 = 2000 kg

Required capacity factor for dowel group = $\frac{2000}{722}$ = 2.77

Spacing of Dowel Bars

Effective distance upto which there is load transfer

= 1.8 *l* = 1.8 × 80 = 144 cm

Assuming a trial spacing of 35 cm between the dowel bars, the capacity factor available for the group

$$= 1 + \frac{144-35}{144} + \frac{144-70}{144} + \frac{144-105}{144} + \frac{144-140}{144}$$

$$= 5 - \frac{350}{144} = 2.57$$

This value of capacity factor available is less than the capacity factor required, i.e., 2.77. Therefore assuming a closer spacing of 30 cm, capacity factor available,

$$= 1 + \frac{144-30}{144} + \frac{144-60}{144} + \frac{144-90}{144} + \frac{144-120}{144} = 2.92$$

As this is greater than the required capacity factor of 2.77, the spacing of the dowel bars is adequate. Therefore provide 2.5 cm dia. Dowel bars of total length 45 cm at 30 cm spacing.

Design of tie bars

Tie bars are used across the longitudinal joints of cement concrete pavements. Tie bars ensure two adjacent slabs to remain firmly together. These bars are not designed to act as load transfer devices. Tie bars are thus designed to withstand tensile stresses, the maximum tensile force in tie bars being equal to the force required to overcome frictional force between the bottom of the adjoining pavement slab and the soil subgrade. The force is considered from the joint location to the subsequent joint or free edge. See Fig. 7.28.

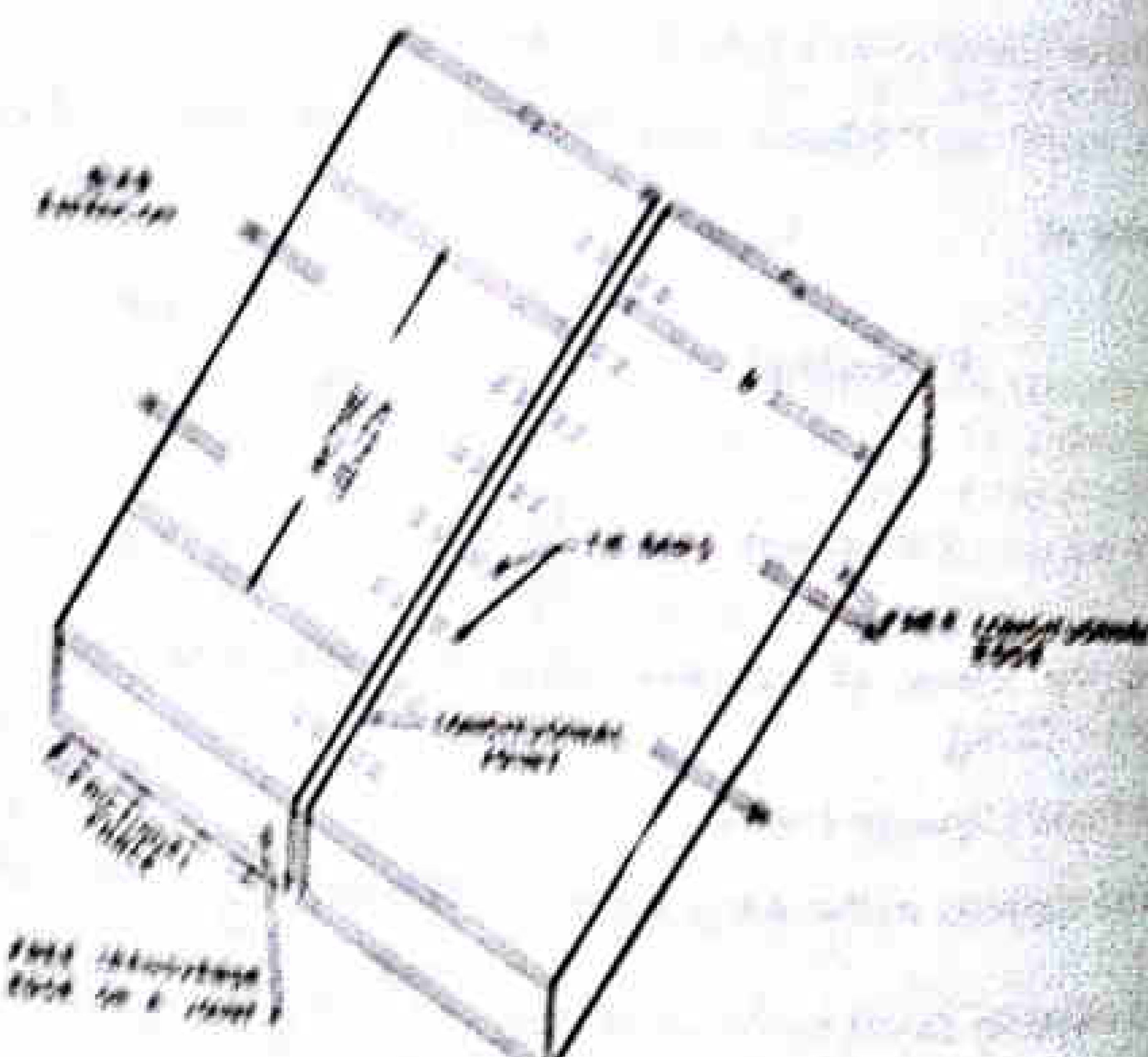


Fig. 7.28 Functioning of Tie Bars

Diameter and Spacing

The diameter and spacing of the tie bars are calculated as explained under :

Area of steel per meter length of joint is obtained by equating the total subgrade friction to the total tension developed in the tie bars.

Thus considering one meter length of the joint,

$$A_s S_s = b \frac{h}{100} W f$$

$$A_s = \frac{b f h W}{100 S_s} \qquad (7.36)$$

where A_s = area of steel per metre length of joint, cm^2

b = distance between the joint and nearest free edge, m

h = thickness of pavement, cm

f = Coefficient of friction between pavement and subgrade (may be taken as 1.5 for rough interface)

W = unit weight of cement concrete, kg/m^3 (2400 kg/m^3)

S_s = allowable working stress in tension for steel, kg/cm^2 (1400 kg/cm^2)

Assuming a suitable diameter of the tie bar (0.8 to 1.5 cm) the spacing of the tie bar can be found to provide the area of steel A_s cm^2 per meter length of the slab.

Length of Tie Bar

The total length of the bar should be atleast twice the length of embedment required on each slab to develop a bond strength equal to the working stress of the steel.

This is obtained from the consideration that the total tensile force developed in tie bar should not exceed the bond strength between tie bar and the concrete. Therefore considering one side of the longitudinal joints,

$$a_s S_s = \frac{L_t}{2} P S_b$$

$$L_t = \frac{2 a_s S_s}{P S_b} \qquad (7.37)$$

Substituting $a_s = \pi d^2/4$ and $P = \pi d$,

$$L_t = \frac{d S_s}{2 S_b}$$

Hence total length of tie bar $= L_t = \frac{2 a_s S_s}{P S_b} = \frac{d S_s}{2 S_b}$ (7.38)

Here, $L_t/2$ = length of tie bar on one side of slab, cm or half length of tie bar

S_s = allowable stress in tension, kg/cm^2 (1400 kg/cm^2)

S_b = allowable bond stress in concrete, kg/cm^2 (this is taken as 24.6 kg/cm^2 for deformed bars and 17.5 kg/cm^2 in plain tie bars).

a_s = cross sectional area of one tie bar, cm^2

P = perimeter of tie bar, cm

d = diameter of tie bar, cm

Example 7.23

As cement concrete pavement has a thickness of 18 cm and has two lanes of 7.2 m with a longitudinal joint along the centre. Design the dimensions and spacing of the tie bar. Use the following data :

$$\text{Allowable working stress in tension, } S_s = 1400 \text{ kg/cm}^2$$

$$\text{Unit weight of concrete, } W = 2400 \text{ kg/m}^3$$

$$\text{Coefficient of friction, } f = 1.5$$

$$\text{Allowable bond stress in deformed bars in concrete, } S_b = 24.6 \text{ kg/cm}^2$$

Solution

Area of steel per metre of longitudinal joint is given by Eq. 7.36.

$$b = \frac{7.2}{2} = 3.6 \text{ m}; h = 18 \text{ cm},$$

$$A_s = \frac{b\,f\,h\,W}{100\,S_s} = \frac{3.6 \times 1.5 \times 18 \times 2400}{100 \times 1750} = 1.333 \text{ cm}^2/\text{m}$$

Using 1 cm diameter bars having area of cross section $a_s = 0.785$ cm^2, the spacing of the tie bars is given by

$$\text{Spacing} = \frac{100 \times 0.785}{1.333} = 58.9 \text{ cm, (say 58 cm)}$$

Using 1 cm diameter tie bars, the length L_t is obtained from Eq. 7.38 and is given by :

$$L_t = \frac{d\,S_s}{2\,S_b} = \frac{1 \times 1400}{2 \times 24.6} = 28.5 \text{ cm}$$

Total length of tie bar = 30 cm, say

Use 1 cm diameter dia bars of length 30 cm at 50 to 58 cm c/c.

7.4.6 IRC Recommendations for Design of Concrete Pavements

(a) Design Parameters

(i) The design wheel load is taken as 5100 kg with equivalent circular area of 15 cm and a tyre inflation pressure ranging from 6.3 to 7.3 kg/cm^2. The traffic volume is projected for 20 years period after construction using the relation :

$$A_d = P' [1 + r]^{(n+20)} \tag{7.39}$$

where A_d = number of commercial vehicles per day (laden weight > 3 tonnes)

P' = number of commercial vehicles per day at last count.

r = annual rate of increase in traffic intensity (may be taken as 7.5% for rural roads if data is not available).

n = number of years between the last traffic count and the commissioning of new cement concrete pavement.

The traffic intensity so obtained is classified and adjustment for the pavement design thickness is made as given in the Table below :

Traffic Classification	Design traffic intensity, A_d (no. of vehicles of wt > 3 tonnes, per day) at the end of design life	Adjustment in design thickness of cc pavement, cm
A	0 to 15	− 5
B	15 to 45	− 5
C	45 to 150	− 2
D	150 to 450	− 2
E	450 to 1500	0
F	1500 to 4500	0
G	> 4500	+2

(ii) The mean daily and annual temperature cycles are collected. The recommended temperature differentials between top and bottom of CC slabs of different thickness at various States and regions in India, for the determination of warping stresses are given in Table 7.4.

Table 7.4 Recommended Temperature Differentials in various Regions of the Country

Zone	State and Regions	Temperature differential to °C in slabs of thickness				
		10 cm	15 cm	20 cm	25 cm	30 cm
I	Punjab, U.P., Rajasthan, Gujrat, Haryana, North M.P., excluding hilly regions and coastal areas.	10.2	12.5	13.1	14.3	15.8
II	Bihar, W. Bengal, Assam and E. Orissa excluding hilly and coastal regions.	14.4	15.6	16.4	16.6	16.8
III	Maharastra, Karnataka, South M.P., A.P., W. Orissa and North T.N. excluding hilly and coastal regions.	14.7	17.3	19.0	20.3	21.0
IV	Kerala, South T.N. excluding hilly and coastal regions.	13.2	15.0	16.4	17.6	18.1
V	Coastal areas bounded by hills	12.8	14.6	15.8	16.2	17.0
VI	Coastal areas unbounded by hills	13.6	15.5	17.0	19.0	19.2

(iii) The modulus of subgrade reaction K is determined using standard plate of 75 cm diameter at 0.125 cm deflection. If 30 cm diameter plate is used, the K-value obtained at 0.125 cm deflection is multiplied by 0.5 in order to estimate the K-value of standard plate diameter. The minimum K-value of 5.5 kg/cm^2 is specified for laying cement concrete pavement. If the K value is lower, a suitable sub-base course may be provided to increase the K-value.

(iv) The flexural strength of cement concrete used in the pavement should not be less than 40 kg/cm^2. As a general guidance the minimum compressive strength on 15 cm cubes may be taken as 280 kg/cm^2 at 28 days and mix design strength of 315 to 350 kg/cm^2, depending upon the degree of quality control. The modulus of elasticity, E and Poisson's ratio, μ may be determined experimentally. The suggested E-value is 3×10^5 kg/cm^2 and $\mu = 0.15$. The coefficient of thermal expansion of concrete may be taken as 10×10^{-6} per °C for design purposes.

(b) Calculation of Stresses

(i) The wheel load stresses at edge region is calculated for the designed slab thickness as per Westergaard's analysis modified by Teller and Sutherland, (Eq. 7.21), using stress chart given in Fig. 7.23.

(ii) Temperature stress at edge region is calculated as per Westergaard's analysis using Bradbury's coefficient given in Fig. 7.25. Equation 7.24 may be used for the calculation.

(iii) Wheel load stress at corner region is calculated as per Westergaard's analysis, modified by Kelley, given in Eq. 7.22 and using the stress chart, Fig. 7.24.

(c) Design Steps for Slab Thickness

(i) The width of slab is decided based on the joint spacings and lane width.

(ii) The length of the CC slab is equal to the spacing of the contraction joints, L_c. This is designed using Eq. 7.30 for plain CC pavements; Eq. 7.31 may be used when reinforcement is provided at the contraction joints for the assumed trial thickness of the slab. However the slab length should conform to the recommendations on *Spacing of Joints* given under Art. 7.4.6 (d).

(iii) A trial thickness value of the slab is assumed for calculating the stresses. The warping stress at edge region is calculated (using Equation 7.26) and this value is subtracted from the allowable flexural stress in concrete to find the residual strength in the pavement to support edge loads.

(iv) The load stress in edge region is found using stress chart, Fig. 7.23 or is calculated (using Eq. 7.21). The available factor of safety in edge load stress with respect to the residual strength is found. If the value of factor of safety is less than 1.0 or is far in excess of 1.0, another trial thickness of the slab is assumed and the calculations are repeated till the factor of safety works out to 1.0 or a slightly higher value for the design thickness *h* cm.

(v) The total stresses at the corner due to wheel load and warping is checked using stress chart Fig. 7.24 and 7.25, (or by using Eq. 7.22 and 7.27) for this thickness *h* cm. If this stress value is less than the allowable, flexural stress in concrete, the slab thickness, h is adequate or else the thickness may be suitably increased (However usually the stresses due to load and warping at the corner region would not be higher than that at edge region).

(vi) The design thickness, *h* is adjusted for the traffic intensity or classification at the end of design life and using the adjustment value from Table to obtain the final adjusted slab thickness.

(d) Spacing of Joints

(i) The maximum spacing recommended for 25 mm wide expansion joints is 140 m when the foundation is rough, for, all slab thicknesses. When the foundation surface is smooth (i.e., surface covered with water proof paper before laying the CC slab) the maximum spacing may be 90 m for slab thicknesses upto 20 cm and spacing of 120 m for slab thickness 25 cm, when the pavement is constructed in summer, however when the pavement is constructed in winter, the above spacings may be restricted to 50 and 60 m respectively.

(ii) The maximum contraction joint spacings may be kept at 4.5 m in unreinforced slabs of all thickness. In the case of reinforced slabs, the contraction joint spacing may be 13 m for 15 cm thick slab with steel reinforcement of 2.7 kg/m^2 and 14 m spacing for 20 cm thick slabs with steel reinforcement of 3.8 kg/m^2.

(e) Design of Dowel Bars

The dowel bar system may be designed on the basis of Bradbury's analysis for load transfer capacity of a single dowel bar in shear, bending and bearing in concrete, using Eq. 7.32, 7.33 and 7.34.

The minimum dowel length is taken as ($L_d + \delta$), the value of L_d is determined using Eq. 7.35. The load capacity of the dowel system is assumed to be 40% of the design wheel load. The distance on either side of the load position upto which the dowel bars are effective in load transfer is taken as 1.8 times the radius of relative stiffness, *l*.

Dowel bars do not function satisfactorily in thin slabs and therefore dowel bars are provided in slab of thickness 15 cm or more. IRC recommends 2.5 cm diameter dowel bars of length 50 cm to be spaced at 20 cm in the case of 15 cm thick slabs and spaced at 30 cm in the case of 20 cm thick slabs, the design load being 5100 kg.

(f) Design of Tie Bars

Tie bars are designed for longitudinal joints as explained in Art. 7.4.5. Permissible bond stress in deformed bars is 24.6 kg/cm^2 and that in plain tie bars is 17.5 kg/cm^2. Allowable working stress in tensile steel is taken as 1400 kg/cm^2. Recommended details of tie bars are given in Table 7.5 :

Table 7.5

Slab thickness, cm	Tie bar details, cm			
	Diameter	Max. spacing	Plain bars	Deformed bars
15	0.8	38	40	30
	1.0	60	45	35
20	1.0	45	45	35
	1.2	64	55	40
25	1.0	30	45	35
	1.2	55	55	40
	1.4	62	65	46

(g) Design of Reinforcement

Reinforcement in CC pavements are intended to prevent deterioration of the cracks and not to increase the flexural strength of uncracked slabs. The area of longitudinal and transverse steel required per metre width or length of slab is computed from the formula :

$$A = \frac{L f w}{2 S} \qquad (7.40)$$

where A = area of steel required per m width or length of the slab, cm^2

L = distance between free transverse joints (for longitudinal steel) or free longitudinal joints (for transverse steel), m

f = coefficient of friction between pavement and subgrade, usually taken as 1.5.

S = allowable working stress in steel, usually taken as 1400 kg/cm^2 or 50 to 60 percent of minimum yield stress, kg/cm^2

w = weight of unit area of pavement slab, kg/m^2

The reinforcement may be placed 5 cm below the surface of the slab and is continued across dummy groove joints to serve the purpose of tie bars. At all full depth joints and edges, the reinforcement is kept atleast 5 cm away from the face of joint or edge.

Example 7.24

Design the following details of a plain cement concrete pavement for a two lane highway.

(a) Spacing of expansion and contraction joints

(b) Pavement slab thickness

(c) Dowel bars for expansion joints

(d) Tie bars for longitudinal joints

Follow the design procedure recommended by IRC where ever applicable. Use the given data, IRC load stress charts for edge and corner regions, and assume any other data not provided here.

Width of expansion joint gap = 2.5 cm

Maximum variation in temperature between summer and winter = 35 °C

Thermal coefficient of concrete = 10×10^{-6} per °C

Allowable tensile stress in CC during curing = 0.8 kg/cm^2

Coefficient of friction = 1.5

Unit weight of CC = 2400 kg/cm^3

Design wheel load = 5100 kg

Radius of contact area = 15 cm

Present traffic intensity = 950 commercial vehicles/day

Modulus of reaction of sub-base course = 8 kg/cm^3

Flexural strength (allowable flexural stress) of concrete = 40 kg/cm^2

E value of concrete = 3×10^5 kg/cm^2

μ value = 0.15

Design load transfer through dowel system = 40%

Permissible flexural stress in dowel bar = 1400 kg/cm^2

Permissible shear stress in dowel bar = 1000 kg/cm^2

Permissible bearing stress in concrete = 100 kg/cm^2

Permissible tensile stress in steel (tie bar) = 1400 kg/cm^2

Permissible bond stress in deformed tie bars = 24.6 kg/cm^2

Temperature differential values in the region :

Slab thickness, cm	15	20	25
Temperature differential in slab in the region, °C	14.6	15.8	16.3

Solution

(a) Joint Spacing

$$\delta' = \frac{1}{2}\text{ joint} = \frac{2.5}{2} = 1.25 \text{ cm}$$

$$\text{Spacing of expansion joint } L_s = \frac{\delta'}{100\,C\,(T_2 - T_1)} = \frac{1.25}{100 \times 10 \times 10^{-6} \times 35} = 35.7 \text{ m}$$

which is less than maximum specified spacing of 140 m and so acceptable. Contraction joint spacing in plain CC,

$$L_c = \frac{2S_c \times 10^4}{W.f} = \frac{2 \times 0.8 \times 10^4}{2400 \times 1.5} = 4.45 \text{ m}$$

which is less than maximum specified spacing of 4.5 m and hence acceptable.

Therefore provide contraction joints at 4.45 m spacing and expansion joints at every 8th such joints i.e., 4.45 × 8 = 35.5 m spacing (instead of 35.7 m).

(b) Pavement Slab Thickness

Assume trial thickness of slab = 20 cm

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

$$= \left[\frac{3 \times 10^5 \times 20^3}{12 \times 8 (1 - 0.15^2)}\right]^{\frac{1}{4}} = 71.1 \text{ cm}$$

$$\frac{L_x}{l} = \frac{445}{71.1} = 6.26$$

$$\frac{L_y}{l} = \frac{350}{71.1} = 4.92$$

From Fig. 7.25 warping stress coefficient C_x at $\frac{L_x}{l}$ of 6.26 = 0.92.

At $L_y/l = 4.92$, $C_y = 0.72 < C_x$

Temperature differential for 20 cm thick slab = 15.8°C

$$\text{Warping stress at edge, } S_{te} = \frac{C_x.E.e.t}{2}$$

$$= \frac{0.92\times3\times10^5\times10\times10^{-6}\times15.8}{2} = 21.8 \text{ kg/cm}^2$$

Residual strength in concrete slab at edge region

$$= 40.0 - 21.8 = 18.2 \text{ kg/cm}^2$$

Load stress in edge region, using IRC stress chart (Fig. 7.23), corresponding to

$$h = 20,\ K = 8,\ S_e = 27.5 \text{ kg/cm}^2$$

$$\text{Factor of safety available} = \frac{\text{residual strength}}{\text{edge load stress}} = \frac{18.2}{27.5} = 0.66$$

As the factor of safety is less than 1.0, it is unsafe. Therefore assume a higher slab thickness say h = 24 cm.

$$l = \left[\frac{3\times10^5\times24^3}{12\times8(1-0.15^2)}\right]^{0.25} = 81.53 \text{ cm}$$

$$L_x/l = \frac{445}{81.53} = 5.46$$

$$C_x = 0.80 \text{ (from chart Fig. 7.25); } C_y \text{ at } L_y/l \text{ of } 4.29 = 0.6$$

Temperature differential for 24 cm thick slab (by interpolation) = 16.2°C

$$S_{te} = \frac{1}{2}\times3\times10^5\times10\times10^{-6}\times16.2\times0.8 = 19.44 \text{ kg/cm}^2$$

Residual strength at the edge

$$= 40.0 - 19.44 = 20.56 \text{ kg/cm}^2$$

Load stress at edge, using stress chart (Fig. 7.23) for

$$h = 4,\ K = 8,\ S_e = 19.2 \text{ kg/cm}^2$$

$$\text{Factor of safety available} = \frac{20.56}{19.2} = 1.07 \text{ which is safe and acceptable value}$$

Therefore provide a tentative design thickness of 24 cm.

Check for corner load stress : Using IRC stress chart Fig. 7.24, for h = 24, K = 8, the value of S_c = 23.0 kg per cm^2.

$$\text{Corner warping stress } S_{tc} = \frac{E.e.t}{3(1-\mu)}\sqrt{\frac{a}{l}}$$

$$= \frac{3\times15^5\times10\times10^{-6}\times16.2}{3(1-0.15)}\sqrt{\frac{15}{81.53}} = 7.1 \text{ kg/cm}^2$$

The worst combination of stresses at the corner is 23.0 + 7.1 = 30.1 kg/cm^2, which is also less than the allowable flexural strength of 40 kg/cm^2 and hence the design is safe.

Adjustment for Traffic intensity

$$A_d = P'[(1+r)]^{(n+20)}$$

Assuming a growth factor r = 7.5% and the number of years after the last count before the new pavement is opened to traffic, n = 3.

$$A_d = 950\left[1+\frac{7.5}{100}\right]^{(n+20)} = 5013 \text{ cv/day}$$

This traffic intensity being in the range > 4500, falls in group G and the adjustment factor is + 2 cm.

Therefore the revised design thickness of the slab

$$= 24 + 2 = 26 \text{ cm}$$

(c) Dowel bars

$$\text{Assume dowel bar diameter} = 2.5 \text{ cm}$$

$$\text{Joint width, } \delta = 2.5 \text{ cm}$$

For Equal capacity in bending and bearing

$$L_d = 5d\left[\frac{F_t}{F_b}\times\frac{(L_d+1.5\,\delta)}{(L_d+8.8\,\delta)}\right]^{\frac{1}{2}}$$

$$= 5\times2.5\left[\frac{1400}{100}\times\frac{L_d+1.5\times2.5}{L_d+8.8\times2.5}\right]^{\frac{1}{2}}$$

By substituting different values of L_d by trials (as in Example 7.22), the value of L_d is found to be 42.2 cm.

$$\text{Length of dowel bar} = L_d + \delta = 42.2 + 2.5 = 44.7 \text{ cm}$$

Therefore provide 45 cm long dowel bars of diameter 2.5 cm

$$\text{Actual value of } L_d = 45.0 - 2.5 = 42.5 \text{ cm}$$

Load transfer capacity of single dowel :

$$P'(\text{shear}) = 0.785\, d^2 F_s$$

$$= 0.785\times2.5^2\times1000 = 4906 \text{ kg}$$

$$P'(\text{bending}) = \frac{2\,d^2 F_t}{L_d+8.8\,\delta} = \frac{2\times2.5^3\times1400}{42.5+8.8\times2.5} = 678 \text{ kg}$$

$$P'(\text{bearing}) = \frac{F_b\,.\,L_d^2\,.\,d}{12.5\,(L_d+1.5\,\delta)} = \frac{100\times42.5^2\times2.5}{12.5\,(42.5+1.5\times2.5)} = 781 \text{ kg}$$

Taking the lowest value for design, P' (design) = 678 kg

Load capacity factor required :

Load capacity of the dowel group $= 5100 \times \frac{40}{100} = 2040$ kg

$$\text{Capacity factor required} = \frac{2040}{678} = 3.0$$

Spacing of dowel bars :

Radius of relative stiffness for revised slab thickness of 24 cm,

$$l = \left[\frac{3 \times 10^5 \times 26^3}{12 \times 8(1-0.15^2)}\right]^{1/4} = 86.6 \text{ cm}$$

Effective distance upto which there is load transfer

$$= 1.8\,l = 1.8 \times 86.6 = 155.9 \text{ cm}$$

Assuming a trial spacing of 35 cm between the dowel bars, the capacity available for the group

$$= 1 + \frac{155.9-35}{155.9} + \frac{155.9-70}{155.9} + \frac{155.9-105}{155.9} + \frac{155.9-140}{155.9}$$

$$= 2.77 < \text{the required value of } 3.0.$$

Assume dowel bar spacing of 30 cm.

$$\text{Capacity factor} = 1 + \frac{155.9-30}{155.9} + \frac{155.9-60}{155.9} + \frac{155.9-90}{155.9} + \frac{155.9-120}{155.9} + \frac{155.9-150}{155.9} = 3.11$$

As this value is greater than the required capacity factor of 3.0, 30 cm spacing of the dowel bars is adequate. Therefore provide 2.5 cm dia. Dowel bars at expansion joints, of total length 45 cm at a spacing of 30 cm centres.

(d) Tie Bars

Area of steel per metre length longitudinal joint,

$$A_s = \frac{b.f.h.W}{100\,S_s} = \frac{3.5 \times 1.5 \times 26 \times 2400}{100 \times 1400} = 2.34 \text{ cm}^2 \text{ per m length}$$

Assuming 1 cm diameter of the bars, cross sectional area of each tie bar $a_s = 0.785\text{cm}^2$.

Perimeter of the tie bar = 3.14 cm

Number of tie bars required per meter length of joint

$$= \frac{A_s}{a_s} = \frac{2.34}{0.785} = 2.98$$

$$\text{Spacing of tie bar} = \frac{100}{2.98} = 33.5 \text{ cm}$$

Provide a spacing of tie bar, say 33 cm

$$\text{Length of plain tie bar, } L_t = \frac{d.S_s}{2S_b} = \frac{1 \times 1400}{2 \times 24.6} = 28.5 \text{ cm}$$

The length of tie bar may be increased by 5 cm for tolerance in placement.

Therefore provide 1 cm diameter deformed tie bars, 34 cm in length at a spacing of 33 cm.

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PROBLEMS

1. Explain 'Flexible and Rigid' pavements and bring out the points of difference.
2. Draw a sketch of flexible pavement cross section and show the component parts. Enumerate the functions and importance of each component of the pavement.
3. What are the various factors to be considered in pavement design ? Discuss the significance of each.
4. Discuss the importance of gross wheel load and contact pressure in stress distribution pattern and in pavement design.
5. Explain ESWL and the concept in the determination of the equivalent wheel load.
6. The loaded weight on the rear dual wheels of a truck is 5500 kg. The centre to centre spacing and clear space in the dual wheels are 30 cm and 10 cm respectively. Calculate the ESWL for pavement thickness (i) 20 cm, (ii) 40 cm, (iii) 70 cm.
7. Discuss the effects of repeated applications of loads on pavements. Explain equivalent wheel load factors for load repetitions.
8. Calculate the design repetitions for ten year period equivalent to 2268 kg wheel load if the mixed traffic in both directions is 1860 vehicles per day. The details of distribution of different wheel loads of commercial vehicles are given below :

Wheel load, kg	Percentage in total traffic volume
2268	25
2722	12
3175	9
3629	6
4082	4
4536	2
4990	1

9. Explain how the elastic modulii of subgrade and base course are estimated using plate bearing test data.
10. Explain how climatic variation affects pavement design and performance.
11. What do you understand by frost action ? Discuss the effects and factors on which the intensity of frost action depends. Suggest measures to prevent or reduce the adverse effects.
12. Enumerate the various methods of flexible pavement design. Briefly indicate the basis of design in each case.
13. Explain group index method of pavement design. What are the limitations of this method ?

14. A subgrade soil sample has the following properties :

    Soil passing soil 0.075 mm sieve = 60%

    Liquid Limit = 55%

    Plastic Limit = 45%

    Design the pavement section by G.I. method for heavy traffic with over 400 commercial vehicles per day.
15. Explain the CBR method of pavement design. How is this method useful to determine thickness of component layers ?
16. Discuss the advantages and limitations of C.B.R. method of design.
17. The CBR value of subgrade soil is 8 percent. Calculate the total thickness of flexible pavement using (i) design curve developed by California State Highway Department (ii) design chart recommended by I.R.C. (iii) design formula developed by the U.S. Corps of Engineers.

    Assume light traffic or 3175 kg wheel load and tyre pressure of 5 kg/cm^2.
18. The C.B.R. test carried out on a subgrade soil gave the following readings :

Penetration, mm	Load, kg	Penetration, mm	Load, kg
0.0	0.0	3.0	58.0
0.5	4.0	4.0	70.0
1.0	14.0	5.0	77.5
1.5	30.0	7.5	93.2
2.0	41.0	10.0	102.5
2.5	50.0	12.5	110.8

The different pavement materials available near the construction site are as follows.

(i) Sandy soil with CBR value = 10%

(ii) Soil-kankar mix with CBR value = 25%

(iii) Broken stone and gravel with CBR = 90%

(iv) Bituminous concrete for surfacing = min. 5 cm thick

Design the pavement structure for commercial vehicles of 2000 per day, with 8.0% growth rate.

19. Explain the California resistance value method of flexible pavement design.
20. Calculate the traffic index value for 10 year period using the following data.

No. of axles	ADT (both directions)
2	700
3	300
4	100
5	20

Assume 60 percent increase in traffic volume in 10 years.

21. Calculate equivalent cohesiometer value for the three pavement component layers given below :