Government of Nepal

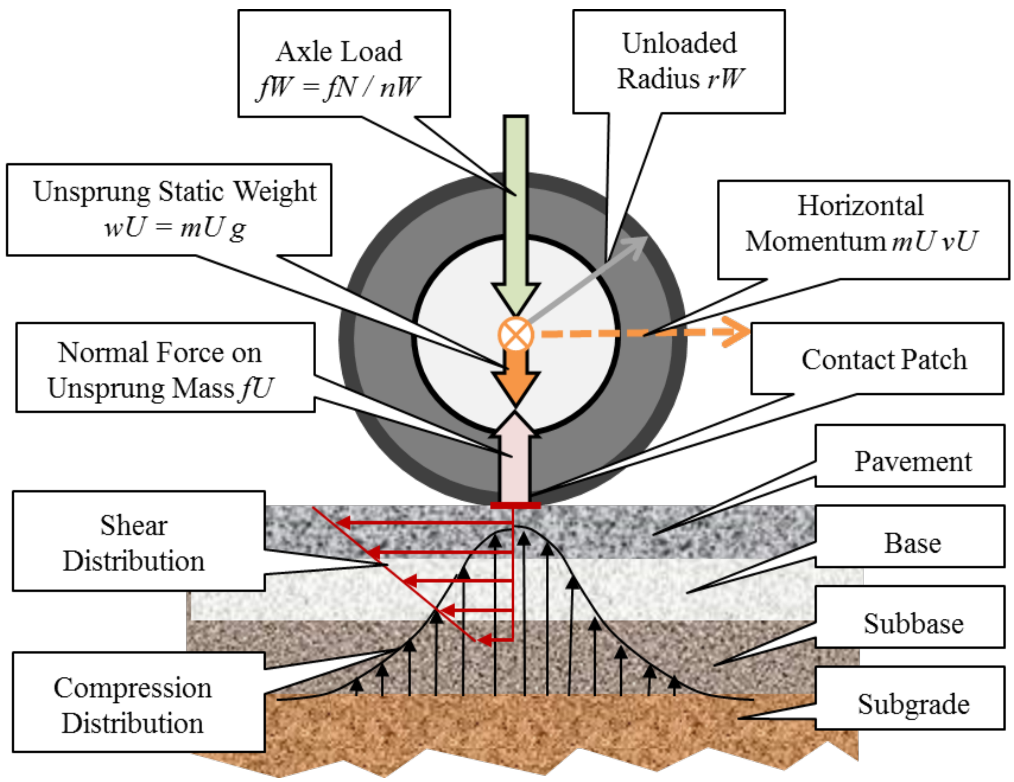
Ministry of Physical Infrastructure and Transport

Department of Roads

Quality, Research and Development Center

Chakupat, Lalitpur

Technical Guidance of Subgrade Evaluation



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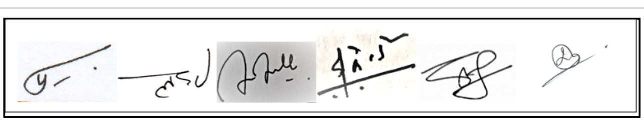

Foreword

Subgrade soil is an integral part of the road pavement structure as it provides the support to the pavement from beneath. When soil is used in embankment construction, in addition to stability, incompressibility is also important as differential settlement may cause failures.

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

This guideline will be useful to all DoR professionals to access the correct CBR of subgrade before design as well as during construction for sustainable pavement performance.

Thank You

……………………

Er. Arjun Jung Thapa
Director General
Department of Roads

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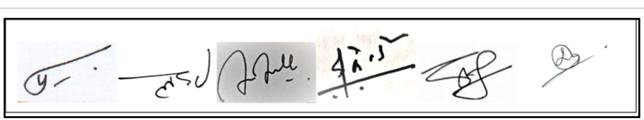

Acknowledgement

The Department of Roads has formed 6-Member Departmental Committee as Er. Prabhat Kumar Jha, SE as Coordinator, Er. Laxmi Datta Bhatt,SDE as Member , Er. Prem Prakash Khatari,SDE as Member , Er. Jibendra Mishra,SDE as Member , Er. Shankar Khanal as Member and Er. Shiv Raj Adhakari as Invitee Member.

The effort and dedication of the 6-Member Departmental Committee for Drafting and Review of guidelines/standards/norms, are highly appreciable.

I believe that the guideline would be helpful to all DoR officilas.

Dr. Bijay Jaisi
Director
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Table of Contents

1. General

The subgrade is the top 500 mm below formation (Lowest level of pavement crust) level , and is made up of in-situ material(Cut road section), select soil or stabilized soil that forms the foundation of a pavement. The support provided by the subgrade is generally regarded as one of the most important factors in determining pavement design thickness, composition and performance. The level of support as characterised by the subgrade strength or modulus is dependent on the soil type, density and moisture conditions at construction and during service.

A subgrade's performance generally depends on two interrelated characteristics:

- Load bearing capacity. The subgrade must be able to support loads transmitted from the pavement structure. This load bearing capacity is often affected by degree of compaction, moisture content, and soil type. A subgrade that can support a high amount of loading without excessive deformation is considered good.
- Volume changes. Most soils undergo some amount of volume change when exposed to excessive moisture or freezing conditions. Some clay soils shrink and swell depending upon their moisture content, while soils with excessive fines may be susceptible to frost heave in freezing areas.

One of the principal objectives of subgrade evaluation is to determine, for design, a subgrade CBR value. A subgrade design CBR is determined for each identifiable unit defined on the basis of topography, drainage and soil type.

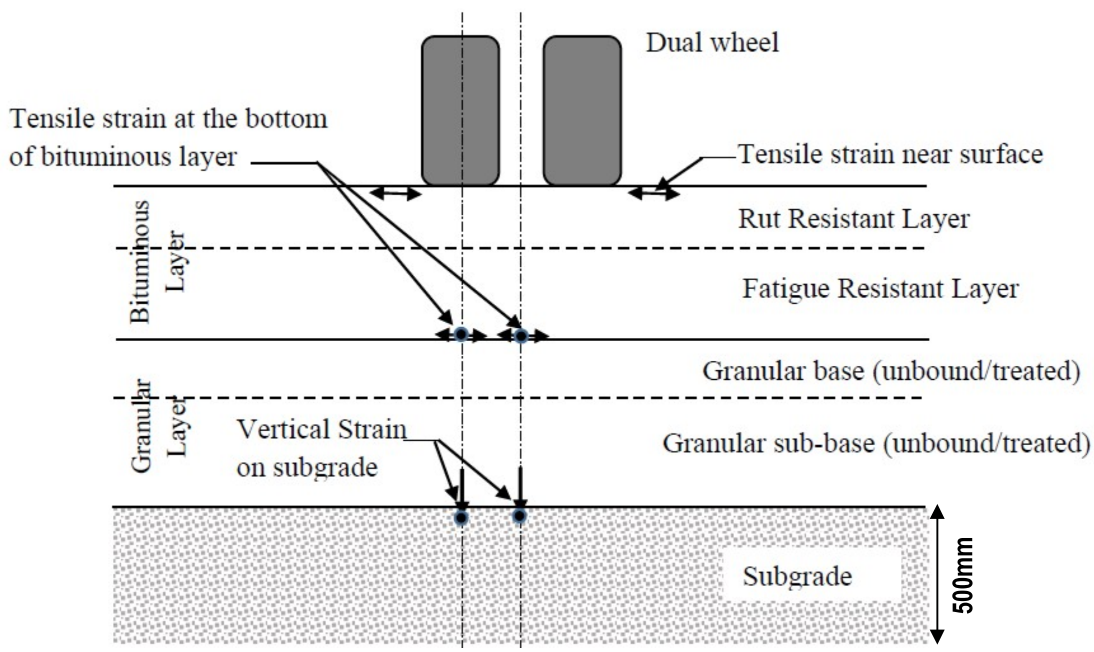
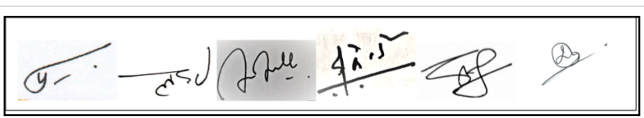


Figure 1.1: Pavement Structure Components



2. Measures of Subgrade Support

The measures of subgrade support are the California Bearing Ratio (CBR), and the elastic parameters – vertical modulus (E_V), horizontal modulus (E_H) and Poisson's ratio (υ).

2.1 California bearing ratio

The California bearing ratio (CBR) is a penetration test for evaluation of the mechanical strength of natural ground, subgrades and base courses beneath new carriageway construction. It was developed by the California Department of Transportation before World War II.

The basic site test is performed by measuring the pressure required to penetrate soil or aggregate with a plunger of standard area. The measured pressure is then divided by the pressure required to achieve an equal penetration on a standard crushed rock material.

The CBR test is conducted one of the following standards :

- ASTM Standards D1883-05 (for laboratory-prepared samples) and D4429 (for soils in place in field), and AASHTO T193.
- BS 1377: Soils for civil engineering purposes: Part 4, Compaction related tests, and in Part 9: In-situ tests.
- IS : 2720 (Part 16) – 1987,Methods of Test for Soil (Part 16 Laboratory Determination of CBR)

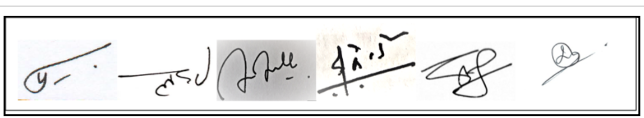
The harder the surface, the higher the CBR rating. A CBR of 3 equates to tilled farmland, a CBR of 4.75 equates to turf or moist clay, while moist sand may have a CBR of 10. High quality crushed rock has a CBR over 80. The standard material for this test is crushed California limestone which has a value of 100, meaning that it is not unusual to see CBR values of over 100 in well-compacted areas.

2.2 Requirements of CBR for Subgrade

Subgrade should be well compacted to limit the scope of rutting in pavement due to additional densification during the service life of pavement. The subgrade has to be compacted to a minimum of 97 per cent of laboratory dry density that is achieved with heavy compaction as per IS: 2720 (Part 8) for Expressways, National Highways, State Highways, District Roads and other heavily trafficked roads.

IRC: 36 "Recommended Practice for the construction of Earth Embankments and Sub-grades for Road Works" shall be followed for guidance during planning and execution of work. The selected soil forming the subgrade must have a minimum CBR of 8 percent for roads having traffic of 450 commercial vehicles per day or higher.

The effective subgrade CBR should be more than 5 % for roads estimated to carry more than 450 commercial vehicles per day (cvpd) (two-way) in the year of construction.



3. Factors to be considered in Estimating Subgrade Support

Many factors must be considered in determining the design support conditions, including:

- Subgrade variability
- Consequences of premature distress (Performance Risk)
- Sequence of earthworks construction
- Target compaction moisture content and field density achieved
- Moisture changes during service life
- Pavement cross-section
- Subsurface drainage and the depth to the water table
- Presence of weak layers below the design subgrade level.

3.1 Subgrade Variability

Subgrades are inherently variable in nature and reflect the changes in topography, soil type, and drainage conditions that generally occur along an existing or proposed road alignment. Hence the selection of a subgrade design value requires adequate consideration of the degree of variability within a particular project section, and the quantity and quality of data on subgrade properties.

3.2 Performance Risk

The investigation methodology and the strength (or modulus) assessment techniques adopted to determine the design support condition should be consistent with the required level of performance risk for the pavement under consideration. More comprehensive testing programs and/or conservative design values are commonly selected when the consequences of premature pavement distress are highly significant or considered unacceptable.

3.3 Sequence of Earthworks Construction

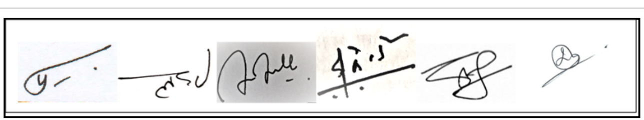
Pre-construction planning can consider the use of selected subgrade materials that result in significant construction savings. The pavement design can be based on the CBR of the selected subgrade material at service moisture and density conditions. Where material selection is not feasible, or where uncertainty exists, a preliminary evaluation of subgrade materials may be necessary, with confirmation at the time of construction. Allowances must be made for any changes in subgrade moisture content that may occur after construction while the pavement is in service.

Even where the design cannot be based on selected subgrade material, good management of the earthworks during construction will ensure that the best available material is used. The careful selection of subgrade materials during pre-construction will result in a more controlled subgrade.

Careful selection of subgrade material beginning at the pre-construction stage also has the significant advantage that it provides a more uniform subgrade. This leads to more uniform pavement configurations, less subgrade testing and more uniform pavement performance.

3.4 Compaction Moisture Content Used and Field Density Achieved

As the strength of subgrade materials is influenced by compaction and moisture content, consideration should be given during design to the likely construction densities and moisture conditions specified for the construction of the subgrade. An indication of the likely effects of variations in relative density and moisture content on subgrades is shown in Figure 3.1.



While compaction of clay subgrades to specified density may be achieved at very low moisture contents, this practice results in an open soil structure which is likely to weaken considerably on wetting. This weakening will be compounded by any density loss due to swelling of the clay.

Volume changes are normally minimized if the subgrade is compacted to the required density at a moisture content consistent with the moisture regime that is expected to prevail most frequently during the design period. However, in areas where the most frequent conditions are very dry, but where wetting may occur, the comments above concerning open soil structure in clays should be noted, and compaction closer to optimum moisture content (OMC) considered.

The OMC for field compaction can differ significantly from the long-term in-service moisture content. This needs to be considered when selecting the moisture conditions for determination of subgrade design CBR.

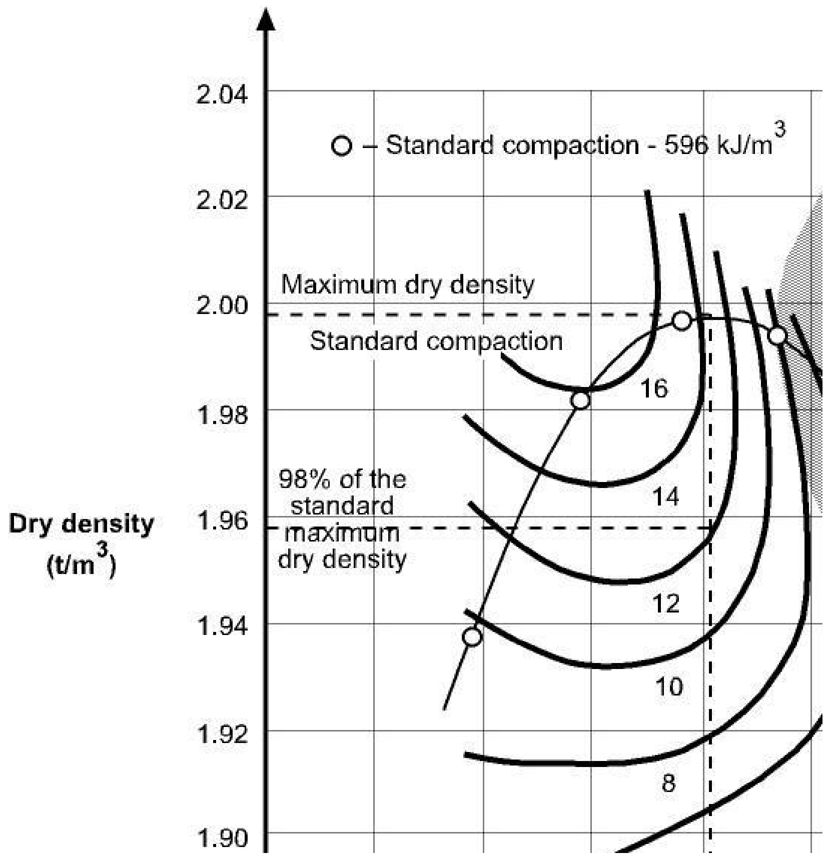


Figure 3.1: Example of variation of CBR with density and moisture content for clayey sand

3.5 Moisture Changes during Service Life

The placing of a sealed pavement surfacing isolates the subgrade from some of the principal influences which affect moisture changes, especially infiltration of large quantities of surface water and evaporation. Where these influences are the controlling ones (i.e. drier environments), the moisture conditions in subgrades generally tend to remain relatively uniform after an initial adjustment period. In such situations, the subgrade under the central region of the pavement is said to reach an equilibrium moisture condition. This region is flanked by two outer regions having moisture conditions that vary with time due to seasonal climatic influences, termed edge effects. Edge effects generally occur under the outer one to two metres of the sealed surfacing. The magnitude of these fluctuations generally increases with distance from the centre of the road towards the edge of the sealed surfacing.

In high rainfall areas, subgrade infiltration – particularly lateral infiltration through unsealed shoulders, through defects in wearing surfaces, or through joints – can have a major influence on the subgrade moisture conditions. Specific action should therefore be taken to guard against this influence.

The proximity of the ground water table or local perched water table to the pavement wearing surface may also play a significant role in influencing the subgrade moisture conditions. In circumstances where the height of the water table fluctuates seasonally, the subgrade moisture condition will reflect these fluctuations equally across the central and peripheral regions of the pavement.

Overall, moisture changes in the subgrade reflect variations in rainfall and temperature which cause changes in water table levels, saturation or drying of the shoulders, changes in ground water seepage, etc.

The type of subgrade soil will often control the rate at which seasonal moisture changes occur, and their extent. For example, sand and silty-sand subgrades may reach their wettest condition a few days after heavy rain occurs, while clay or silty-clay subgrades may not reach their wettest condition until months after the end of the wet season. Similarly, sand and silty-sand subgrades will saturate readily, being more permeable, whereas long periods of access to water are required to substantially change the moisture condition of a highly plastic clay subgrade.

Wetting-up of subgrades can, however, be accelerated by an active head due to ponded water or a ground water seepage regime, or by cracking in the subgrade soils. Drying of subgrades, particularly of clays, can be delayed by surface tension effects and the availability of water vapour in the pores. A combination of this accelerated wetting and delayed drying often results in a progressive wetting up of the subgrade; commonly to or beyond optimum moisture content, even in 'dry' areas.

3.6 Expansive soils

Loss of pavement shape due to moisture changes in expansive soils can be a significant factor in the need to rehabilitate pavements. A guide to the identification and qualitative classification of expansive soils is presented in Table 3.1. The swell test is preferred to the plasticity index test if facilities are available.

Table 3.1: Guide to classification of expansive soils

Expansive nature	Liquid limit (%)	Plasticity Index	PI x % < 0.425 mm	Swell (%) (1)
Very high	> 70	> 45	> 3200	> 5.0
High	> 70	> 45	2200–3200	2.5–5.0
Moderate	50–70	25–45	1200–2200	0.5–2.5
Low	< 50	< 25	< 1200	< 0.5

(1) Swell at OMC and 98% MDD using standard compactive effort; four-day soak. Based on 4.5 kg surcharge.

Volume changes in highly expansive soils can be minimized by adoption of some, or all, of the following strategies:

- Construct the subgrade or fill material at a time when its soil suction (the ability of a soil to attract moisture) is likely to be near the long-term equilibrium value.
- Compact the soil at its **equilibrium moisture content** (EMC). This value occurs when a soil is at its equilibrium soil suction value.
- Provide a low-permeability lower subbase or a select fill capping layer above the expansive soil. The minimum thickness of this layer should be the greater of 150 mm or two-and-a-half times the maximum particle size. This capping layer should extend at least 500 mm past the edge of pavement, and if provided, past the kerb and channel, to reduce edge movement.
- Provide a minimum cover of material over the expansive soil for all pavement types. Material used to provide this layer should have swells of less than 1.5% for the top 300 mm and less than 2.5% for the remaining thickness and be placed at an appropriate moisture content to remain within this limit. The required thickness of cover increases with the traffic loading to reflect the better ride quality required on higher traffic volume roads.

- Ensure that the location of pavement drains is confined to the impermeable subbase/select fill capping layer and does not extend into the expansive soils. Drains located within expansive soils will cause fluctuations in the moisture content of the soil.
- Restrict the planting of shrubs and trees close to the pavement.
- Provide – through appropriate design of the cross-section of the road – sealed shoulders and impermeable verge material. A seal width of one to 1.5 m is required outside the edge of the traffic lanes to minimize subgrade moisture changes under the outer wheel path.
- Use appropriate construction techniques when placing the expansive soil.
- Incorporate lime stabilization to reduce the plasticity and increase the volume stability of the upper layer of the expansive clay subgrade.

3.7 Pavement Cross-section and Subsurface Drainage

Features such as width of sealing, boxed construction, relative permeability of pavement layers and the presence and extent of pavement drainage can all have a considerable effect on subgrade moisture conditions and strength.

The outer regions of the pavement and subgrade are subject to significant moisture changes. If this zone of significant moisture fluctuation can be removed from the trafficked area by using sealed shoulders, the more stable moisture conditions may be allowed for in the selection of a design CBR.

Cross-section types with relatively high permeability pavement materials either 'boxed' into the surrounding natural materials or flanked by less permeable shoulder materials can inhibit drainage unless appropriate pavement drainage is provided.

These factors must be considered when deciding how to divide the total road length into homogeneous sub- sections for design purposes. The sub-sections should be selected on the basis that the condition and type of the subgrade material is essentially likely to be constant. They can then form the basis for determining the design subgrade conditions.

3.8 Presence of Weak Layers below the Design Subgrade Level

Evaluation of the actual support provided to the pavement structure by the subgrade can be complicated by the strength variations that often occur with depth. It is essential that the potential effects of any weak layers below the design subgrade level are considered in the pavement design process, particularly for low-strength materials occurring to depths of about 1 m.

Where strength decreases with depth, the subgrade may be sublayered for the purposes of the mechanistic -empirical pavement design of flexible pavements. For subgrade strengths that are constant or improve with depth, the support at the design subgrade level governs the pavement design.

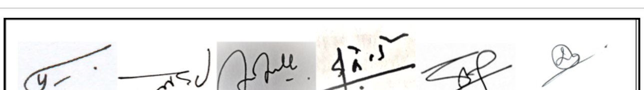
Stabilised Subgrades

Stabilisation can be used to:

- increase the strength and improve the uniformity of subgrades and pavement materials
- provide resistance to the effects of water ingress
- provide a working platform for subsequent construction
- optimise the use of available pavement materials
- reduce layer thicknesses compared to unbound materials.

For the purposes of pavement design, subgrade material which has been stabilised should not generally be assigned a CBR value greater than 15%.

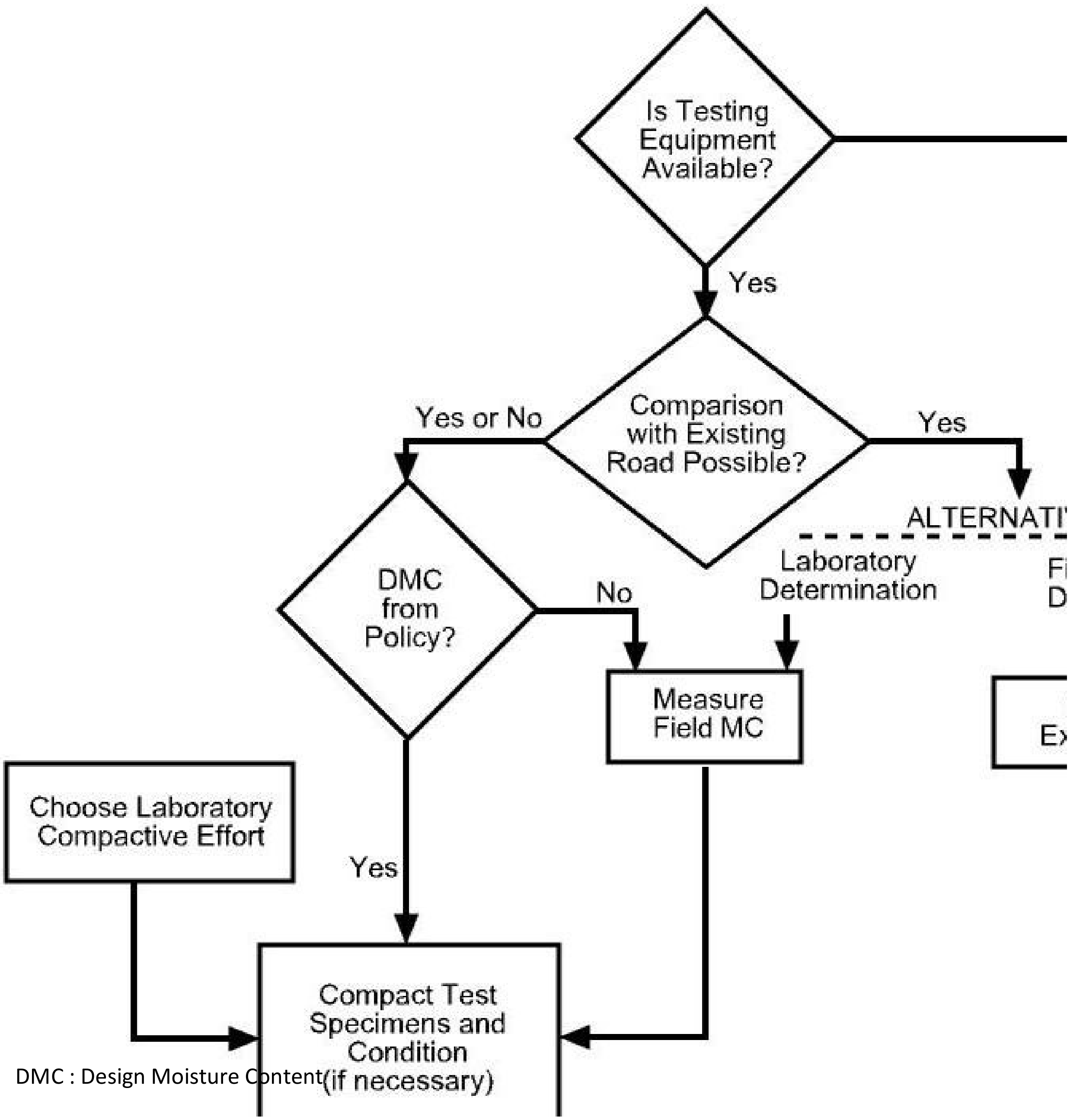
The introduction of heavy duty, purpose-built stabilisation equipment, and the introduction of slow-setting binders, has allowed the construction of thicker stabilised layers with longer working time and reduced shrinkage cracking. This has widened the range of applications to which stabilisation can be applied, particularly for rehabilitation works.



4. Methods for Determining Subgrade Design CBR Value

One of the principal objectives of subgrade evaluation is to determine, for design, a subgrade CBR value at the density and moisture conditions which are expected to prevail in-service for the long-term. A subgrade design CBR is determined for each identifiable unit defined on the basis of topography, drainage and soil type.

As the moisture conditions at the time of construction usually differ from those used in the determination of the design CBR, it is seldom appropriate for in situ field testing during construction to increase the design CBR previously assessed unless a comprehensive investigation concludes the materials encountered are significantly better. Nevertheless, during construction the subgrade is exposed and this often provides an opportunity to confirm that the design CBR has not been overestimated by sampling the full range of subgrade materials and undertaking laboratory testing at appropriate field densities and moisture conditions. In situations where in situ CBR testing is considered appropriate and reliable, this can also be used to confirm or reduce the design CBR value.



DMC : Design Moisture Content

Figure 4.1: Methods for estimating subgrade support values

There are primarily two modes of testing available for estimating subgrade support values: field testing and laboratory testing.

- Field testing is applicable to situations where the support values from the in situ subgrade soil conditions are expected to be similar to those of the proposed pavement.
- Laboratory testing is applicable in both situations i.e. a) as mentioned above and b) when subgrade support is to be determined from first principles. Due consideration should be given to the sample density, moisture, and soaking conditions which simulate the expected pavement support while in-service. The two modes are illustrated in Figure 4.1.

There is a range of direct and indirect testing methods that can provide CBR of the subgrade. Many of these are based on empirical correlations that have considerable variability. For this reason, where possible, a combination of test methods should be used to allow appropriate checks and for confirmation of critical support determinations.

5. Laboratory Determination of Subgrade CBR and Elastic Parameters

Laboratory procedures may be used to determine design CBR or modulus when sufficient samples of the subgrade material for the new pavement can be obtained for detailed laboratory investigations and where a reasonable estimate can be made of likely subgrade density and moisture conditions in-service. The method is particularly useful where there is not a close similarity in material type, density and moisture content between the proposed subgrade and any existing site that may be available for in situ testing.

The test may be performed on undisturbed specimens and on remoulded specimens which may be compacted either statically or dynamically.

Laboratory tests may be undertaken on specimens tested at a density which corresponds to those likely to occur in-service or at a particular compaction standard and moisture as a characterising test. Alternatively, undisturbed samples can be obtained from the field by coring.

It may not always be practicable to prepare laboratory specimens at the selected density. In these cases, at least four specimens should be prepared at densities as close as possible to the characteristic value. The subgrade CBR of the material can then be determined from interpolation of the results for these specimens.

Special pre-treatments may be necessary or desirable when dealing with particular types of subgrade material. For example, for extremely weathered and highly weathered rocks such as siltstone and shale, the effects of construction should be simulated either by applying repeated cycles of compaction of the material to simulate construction, or by introducing other forms of pre-treatment prior to compacting the specimens for testing.

If the testing interval and data are unbiased, and the variability of test results is low, then statistical analysis can be used to determine a design CBR at an appropriate percentile level. To ensure homogeneous sub-sections of subgrade, the CBR values should have a coefficient of variation (i.e. standard deviation divided by the mean) of 0.25 or less. The ten percentile level (i.e. 90% of results exceed this level) is commonly adopted as the design CBR of highway pavements. For roads in arid climates, or roads of lesser importance, higher percentile values may be appropriate (VicRoads 2013 and 2017).

For thickness design purposes using mechanistic-empirical procedures, subgrade materials are assumed to be elastic and cross-anisotropic. A cross-anisotropic material is characterised by five parameters – two moduli (vertical, horizontal), and two Poisson's ratios (vertical and horizontal) and the additional stress parameter (f). In this Part, the ratio of vertical to horizontal modulus is assumed to be 2 and both Poisson's ratios are assumed to be equal. The stress parameter can be determined using the following relationship (Equation 5.1).

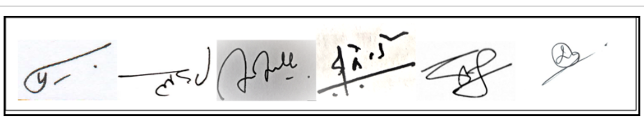
$$f = \frac{Vertical\ Modulus}{1 + Poisson's\ Ratio}$$ **Equation 5.1**

Hence, the values of the five parameters can be determined from the vertical modulus and Poisson's ratio data. The vertical modulus of subgrade can be determined from laboratory testing of conditioned specimens (Thompson & Robnett 1976) or by using the empirical relationship (Equation 5.2).

Modulus (MPa) = 10 × CBR **Equation 5.2**

This equation 5.2 is, at best, an approximation and modulus has been found to vary in the range 5 × CBR to 20 × CBR (Sparks & Potter 1982). This equation may overestimate the subgrade moduli for soils with relatively high CBRs and hence should only be used for CBR ≤ 15. Representative values of Poisson's ratio for subgrades are 0.45 for cohesive materials and 0.35 for non-cohesive materials.

The IRC 37-2018 has suggested the relation between resilient modulus and the effective CBR is given by equation 5.3.



$$\left.\begin{array}{lll} M_R(MPa) = 10 \times CBR & for & CBR \leq 5\% \\ M_R(MPa) = 17.6 \times CBR^{0.64} & for & CBR > 5\% \end{array}\right\}$$ **Equation 5.3**

M_R can be determined in the laboratory by conducting tests as per procedure specified in AASHTO 307-99(2003). For the purpose of design, the resilient modulus (MRS), thus estimated, shall be limited to a maximum value of 100 MPa.

Few Subgrade Strength/Stiffness Correlation Equations

Heukelom & Klomp (1962)

M_R = (1500)(CBR) **Equation 5.4**

Limitation : Only for fine-grained non-expansive soils with a soaked CBR of 10 or less.

1993 AASHTO Guide

M_R = 1,000 + (555)(R-value) **Equation 5.5**

Limitation : Only for fine-grained non-expansive soils with R-values of 20 or less.

AASHTO 2002 Design Guide

M_R= 2555 x CBR0.64 **Equation 5.6**

Limitation : A fair conversion over a wide range of values

5.1 Determination of Density for Laboratory Testing

The density selected for testing should correspond to that which will occur in-service and may be one of the following:

- in situ density of undisturbed or reworked subgrade as appropriate
- minimum standard of compaction achieved in construction (embankments)
- density after swelling has occurred (expansive soils).

5.2 Determination of Moisture Conditions for Laboratory Testing

The designer should ensure, either on the basis of knowledge of moisture conditions likely to occur in the locality, or by means of detailed field investigations, that the laboratory test conditions realistically represent in-service moisture conditions. In many situations, testing under soaked conditions may be warranted.

Several combinations of specimen preparation moisture contents and soaking conditions used in laboratory CBR testing to achieve moisture contents similar to in-service pavements are presented in Table 5.1.

Table 5.1: Typical moisture conditions for laboratory CBR testing

Median annual rainfall (mm) (50 percentile value)	Specimen compaction moisture content	Testing condition	
		Excellent to good drainage	Fair to poor drainage
< 500	OMC	Unsoaked to 4-day soak	1 to 4-day soak
500–800	OMC	Unsoaked to 4-day soak	4 to 7-day soak
> 800	1 to 1.15 × OMC	Unsoaked to 4-day soak	4 to 10-day soak

5.3 California Bearing Ratio Lab Test

This section covers the laboratory method for the determination of C.B.R. of undisturbed and remoulded /compacted soil specimens, both in soaked as well as unsoaked state.

The following table gives the standard loads adopted for different penetrations for the standard material with a C.B.R. value of 100%

Table : 5.3 Standard Loads with respect to the plunger penetration

Penetration of plunger(mm)	Standard load(kg), P_S
2.5	1370
5.0	2055
7.5	2630
10.0	3180
12.5	3600

Equipments and tool required:

- Cylindrical mould with inside dia 150 mm and height 175 mm, provide height and a detachable perforated base plate 10 mm thick.
- Spacer disc 148 mm in dia and 47.7 mm in height along with handle.
- Metal rammers. Weight 2.6 kg with a drop of 310 mm (or) weight 4.89
- Weights. One annular metal weight and several slotted weights we central hole 53 mm in diameter.
- Loading machine. With a capacity of at least 5000 kg and equipped w uniform rate of 1.25 mm/min. Complete with load indicating device.
- Metal penetration piston 50 mm dia and minimum of 100 mm in length
- Two dial gauges reading to 0.01 mm.
- Sieves. 4.75 mm and 20 mm I.S. Sieves.
- Miscellaneous apparatus, such as a mixing bowl, straight edge, scales soaking tank or pan, drying oven, filter paper and containers.

Fig.5.1 Lab-CBR Test Setup

Preparation of test specimen

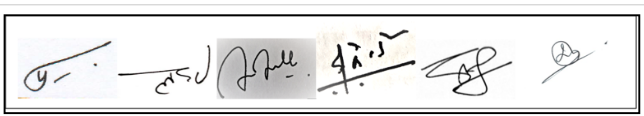
The test may be performed on:

a) undisturbed specimens, and

b) remoulded specimens which may be compacted either statically or dynamically.

The static method of compaction gives the required density but requires considerable pressure and there is a possibility of the actual density varying with depth though the mean density may be the one desired.

Undisturbed Specimens - Undisturbed specimens have to be obtained by fitting to the mould, the steel cutting edge of 150 mm internal diameter and pushing the mould as gently as possible into the ground. This process may be facilitated by digging away soil from the outside as the mould is pushed in. When the mould is sufficiently full of soil, it has to be



removed by under digging, the top and bottom surfaces are then trimmed flat so as to give the required length of specimen ready for testing. If the mould cannot be pressed in, the sample may be collected by digging at a circumference greater than that of the mould and thus bringing out a whole undisturbed lump of soil. The required size of the sample to fit into the test mould has to be then carefully trimmed from this lump. If the specimen is loose in the mould, the annular cavity have to be filled with paraffin wax thus ensuring that the soil receives proper support from the sides of the mould during the penetration test.

The density of the soil has to be determined either by weighing the soil with mould when the mould is full with the soil, or by measuring the dimensions of the soil sample accurately and weighing or by measuring the density in the field in the vicinity of the spot at which the sample is collected in accordance with the method specified in IS : 2720 (Part 28) - 1974 or IS : 2720 (Part 29) - 1975. In all cases, the water content shall be determined in accordance with IS : 2720 (Part 2)-1973.

Remoulded Specimens - The dry density for a remoulding has to be either the field density or the value of the maximum dry density estimated by the compaction tests as per IS : 2720 (Part 7) 1980, and IS : 2720 (Part 8)-1983, or any other density at which the bearing ratio is desired. The water content used for compaction should be the optimum water content or the field moisture as the case may be.

The material used in the remoulded specimen has to pass a 20 mm IS Sieve. Allowance for larger material has to be made by replacing it by an equal amount of material which passes a 20 mm IS Sieve but is retained on 4.75-mm IS Sieve.

Statically Compacted Specimens - The mass of the wet soil at the required moisture content to give the desired density when occupying the standard specimen volume in the mould has to be calculated. A batch of soil has to be thoroughly mixed with water to give the required water content. The correct mass of the moist soils have to be placed in the mould and compaction obtained by pressing in the displacer disc, a filter paper being placed between the disc and the soil.

Dynamically Compacted Specimen - For dynamic compaction, a representative sample of the soil weighing approximately 4.5 kg or more for fine-grained soils and 5.5 kg or more for granular soils have to be taken and mixed thoroughly with water. If the soil is to be compacted to the maximum dry density at the optimum water content determined in accordance with IS : 2720 (Part 7 – 1980) or IS : 2720 (Part 8)- 1983, the exact mass of soil required has to be taken and the necessary quantity of water added so that the water content of the soil sample is equal to the determined optimum water content.

The mould with the extension collar attached has to be clamped to the base plate. The spacer disc has to be inserted over the base plate and a disc of coarse filter paper placed on the top of the spacer disc. The soil-water mixture has to be compacted into the mould in accordance with the methods applicable to the 150 mm diameter mould specified in IS : 2720 (Part 7) - 1980 or IS : 2720 (Part 8) -1983. If other densities and water contents are desired, they may be used and indicated in the report.

The extension collar has to then be removed and the compacted soil carefully trimmed even with the top of the mould by means of a straightedge. Any hole that may then, develop on the surface of the compacted soil by the removal of coarse material, has to be patched with smaller size material; the perforated base plate and the spacer disc has to be removed, and the mass of the mould and the compacted soil specimen recorded. A disc of coarse filter paper has to be placed on the perforated base plate, the mould and the compacted soil has to be inverted and the perforated base plate clamped to the mould with the compacted soil in contact with the filter paper.

In both cases of compaction, if the sample is to be soaked, representative samples of the material at the beginning of compaction and another sample of the remaining material after compaction has to be taken for determination of water content. Each water content sample has to weigh not less than about 50 g.

If the sample is not to be soaked, a representative sample of material from one of the cut-pieces of the material after penetration has to be taken to determine the water content. In all cases, the water content has to be determined in accordance with IS : 2720 (Part 2)-1973.

Procedure

Test for Swelling : A filter paper has to be placed over the specimen and the adjustable stem and perforated plate has to be placed on the compacted soil specimen in the mould. *Weights to produce a surcharge equal to the weight of base material and pavement to the nearest 2.5 kg shall be placed on the compact soil specimen*. The whole mould and weights has to be immersed in a tank of water allowing free access of water to the top and bottom of the specimen. The tripod for the expansion measuring device has to be mounted on the edge of the mould and the initial dial gauge reading recorded. This set-up has to be kept undisturbed for 96 hours noting down the readings every day against the time of reading. A constant water level has to be maintained in the tank through-out the period.

At the end of the soaking period, the change in dial gauge has to be noted, the tripod removed and the mould taken out of the water tank.

The free water collected in the mould has to be removed and the specimen allowed to drain downwards for 15 minutes. Care has to be taken not to disturb the surface of the specimen during the removal of the water. The weights, the perforated plate and the top filter paper have to be removed and the mould with the soaked soil sample has to be weighed and the mass recorded.

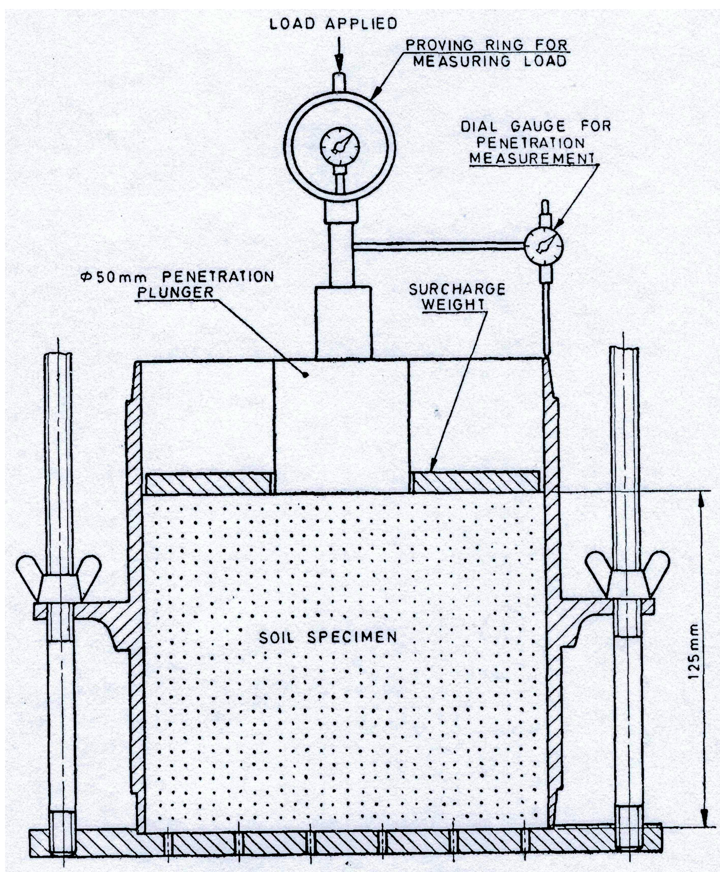


Figure 5.2 Schematic Diagram of Lab-CBR Testing Arrangement

Penetration Test (see Fig. 5.2) - The mould containing the specimen, with the base plate in position but the top face exposed, has to be placed on the lower plate of the testing machine. Surcharge weights, sufficient to produce an intensity of loading equal to the weight of the base material and pavement has to be placed on the specimen. If the specimen has been soaked previously, the surcharge has to be equal to that used during the soaking period. To prevent upheaval of soil into the hole of the surcharge weights, 2.5 kg annular weight shall be placed on the soil surface prior to seating the penetration plunger after which the remainder of the surcharge weights has to be placed. The plunger has to be seated under a load of 4 kg so that full contact is established between the surface of the specimen and the plunger. The load and deformation gauges has to be then set to zero (In other words, the initial load applied to the plunger has to be considered as zero when determining the load penetration relation). Load has to be applied to the plunger into the soil at the rate of 1.25 mm per minute. Reading of the load has to be taken at penetrations of 0.5, 1.0, 1.5, 2.0, 2.5, 4.0, 5.0, 7.5, 10.0 and 12.5 mm (The maximum load and penetration shall be recorded if it occurs for a penetration of less than 12.5 mm). The plunger has to be raised and the mould shall be detached from the loading equipment. About 20 to 50 g of soil has to be collected from the top 30 mm layer of the specimen and the water content shall be determined according to IS : 2720 (Part 2)-1973. If the average water content of the whole specimen is desired, water content sample has to be taken from the entire depth of the specimen. The undisturbed specimen for the test should be carefully examined after the test is completed for the presence of any oversize soil particles which are likely to affect the results if they happen to be located directly below the penetration plunger. The penetration test may be repeated as a check test for the rear end of the sample.

Record of observations

Specimen Data - The specimen data shall be recorded on the data sheet as shown below. Apart from soil identification, etc, this includes The condition of the specimen at the time of testing, type of compaction adopted, the amount of soil fraction above 20 mm that has been replaced and the water content and density determinations before and after the mould has been subjected to soaking.

CALIFORNIA BEARING RATIO TEST

SPECIMEN DATA

PROJECT:	TEST NO:
SAMPLE NO:	DATE
SOIL IDENTIFICATION:	TEST BY:
CONDITION OF SPECIMEN AT TEST:	UNDISTURBED/REMOULDED/SOAKED/UNSOAKED
TYPE OF COMPACTION	Static/Dynamic Compaction
	Light/Heavy Compaction

Soil fraction above 20 mm Replaced….............. Kg

Water Content	Before Soaking Field	After Test		
		Top	Centre	Bottom
Can No.				
Wt of Can +Wet Soil(g)				
Wt of Can +Dry Soil(g)				
Wt of water(g)				
Wt of can(g)				
Wt of dry Soil(g)				
Water content(%)				

Condition of Specimen	Before Soaking	After Soaking
Wt of mould + soil(Kg)		
Wt of mould(Kg)		
Wt of soil(Kg)		
Volume of the specimen(cc)		
Bulk density(g/cc)		
Average water content(%)		
Dry density(g/cc)		

Penetration Data - The readings for the determination of expansion ratio and the load penetration data shall be recorded in the data sheet as shown below.

SOIL MECHANICS LABORATORY

CALIFORNIA BEARING RATIO TEST

PENETRATION DATA

Surcharge Weight Used = …....Kg

Test 1			Test 2	
Penetration	**Load Measuring Device Reading**	**Load (Kg)**	**Load Measuring Device Reading**	**Load (Kg)**

CBR of Specimen at 2.5 mm Penetration =

CBR of Specimen at 5 mm Penetration =

CBR of Specimen = …....%

EXPANSION RATIO

Surcharge weight used (Kg) =

Period of soaking (Days) =

Initial height of specimen, h (mm) =

Initial dial gauge reading, d_s (mm) =

Final dial gauge reading, d_f(mm) =

Expansion ratio = $\frac{d_f - d_s}{h} \times 100$

Remarks

…………………………………………………………………………………………

Calculation

Expansion Ratio - The expansion ratio based on tests conducted has to be calculated as follows:

$$Expansion.Ratio = \frac{d_f - d_s}{h} \times 100$$ **Equation 5.7**

where

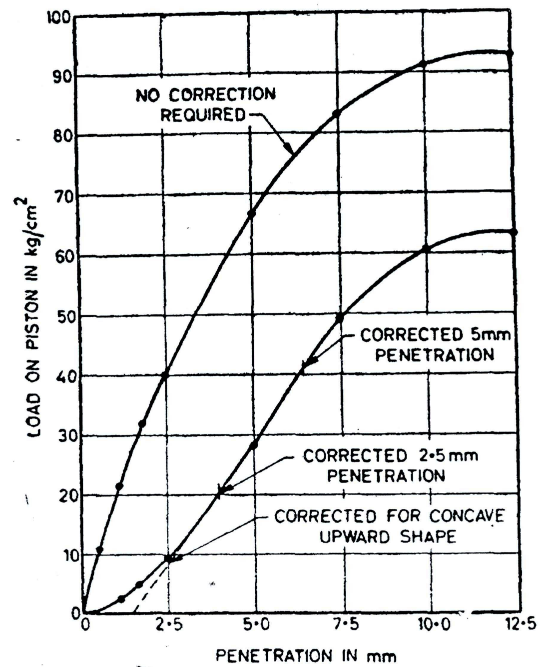
d_f = final dial gauge reading in mm,

d_s = initial dial gauge reading in mm, and

h = initial height of the specimen in mm.

The expansion ratio is used to qualitatively identify the potential expansiveness of the soil.

Load Penetration Curve - The load penetration curve has to be plotted (see Fig. 5.3). This curve is usually convex upwards although the initial portion of the curve may be convex downwards due to surface irregularities. A correction has to be then applied by drawing a tangent to the point of greatest slope and then transposing the axis of the load so that zero penetration is taken as the point where the tangent cuts the axis of penetration. The corrected load-penetration curve would then consist of the tangent from the new origin to the point of tangency on the re-sited curve and then the curve itself, as illustrated in Fig. 5.3.



Penetration Depth (1) mm	*Unit Standard Load* (2) kg/cm²	*Total Standard Load* (3) kgf
2·5	70	1 370
5·0	105	2 055

Figure 5.3 Load verses Penetration Graph

California Bearing Ratio - The CBR values are usually calculated for penetrations of 2'5 and 5 mm. Corresponding to the penetration value at which the CBR values is desired, corrected load value shall be taken from the load penetration curve and the CBR calculated as follows:

$$CBR = \frac{P_T}{P_S} \times 100\% \qquad \textbf{Equation 5.8}$$

where

P_T=corrected unit (or total) test load corresponding to the chosen penetration from the load penetration curve, and

P_S=unit (or total) standard load for the same depth of penetration as for P_T taken from the table 5.3.

Presentation of results

The results of the CBR test are presented as the CBR value and the expansion ratio. The C.B.R. values are usually calculated for penetration of 2.5 mm and 5 mm. Generally the C.B.R. value at 2.5 mm penetration will be greater than that at 5 mm penetration and in such case the former shall be taken as C.B.R. for design purpose. If C.B.R. for 5 mm exceeds that for 2.5 mm, the test should be repeated. If identical results follow, the C.B.R. corresponding to 5 mm penetration should be taken for design. **The maximum permissible variation within the CBR values of the three specifications should be indicated in Table 5.4**

Table 5.4 : Permissible Variation in CBR Value

CBR %	**Maximum Variation in CBR Value**
5	**±1**
5-10	**±2**
11-30	**±3**
31 and above	**±5**

Where variation is more than the above, the average CBR should be the average of test results from at least six samples and not three.

6. Field Determination of Subgrade CBR

This procedure may be used to determine the subgrade CBR in situations where soils similar to those of the subgrade of the road being designed have existed under a sealed pavement for at least two years and are at a density and moisture condition similar to those likely to occur in service. Where further disturbance of exposed subgrade soils on new alignments is unlikely, field CBR testing may also be relevant. In both situations, care must be taken when carrying out the tests that the subgrade is in a critical moisture condition; otherwise, seasonal adjustments may need to be made. A number of field tests may be used to estimate subgrade CBR, e.g. in situ CBR test or cone penetrometer.

If the testing interval and data are unbiased, and the variability of test results is low, then statistical analysis can be used to determine a design CBR at an appropriate percentile level. To ensure homogeneous sub-sections of subgrade, the CBR values should have a coefficient of variation (i.e. standard deviation divided by the mean) of 0.25 or less. The ten percentile level (i.e. 90% of results exceed this level) is commonly adopted for the design of highway pavements. For roads in arid climates, or roads of lesser importance, higher percentile values may be appropriate (VicRoads 2013 and 2017). VicRoads (1995) provides guidance on field testing of subgrades.

6.1 In situ CBR Test

The insitu CBR test should be carried out in accordance with Methods of test for soils, Part 31 Field Determination of California Bearing Ratio IS 2720(Part 31):1990. This test is time-consuming and expensive and its best application is usually as a supplement to other forms of testing.

Apparatus

Loading Device : A mechanical screw loading jack with swivel head for applying load to the penetration piston. The device should have an arrangement for attachment to truck, tractor, truss or any other equipment used to provide load reaction. The jack should be such that a uniform penetration rate of 12.5 mm /min can be achieved. The capacity of the jack should not be less than 50 kN.

Equipment for Providing Reaction for Loading: Truck, tractor, truss or any other suitable equipment. If truck or tractor, is used they should be loaded suitably to give the necessary reaction. If truss is used it should be suitably anchored.

Jacks : Two track-type jacks of 50 to 120 kN capacity, having double acting combination trip and automatic lowering in cases where loaded truck or tractor is used for providing the necessary reaction.

Proving Ring : One calibrated proving ring of suitable capacity having an accuracy of not more than one percent of the anticipated load shall be used. The calibration of the proving ring shall be checked periodically at least once a year.

Metal Penetration Piston: 50 ±0.1 mm in diameter and not less than 100 mm long.

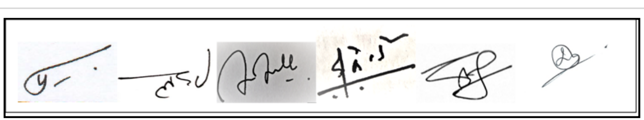
Extensions : Internally threaded pipe or rod extensions not less than 200 cm long furnished in the following

quantities and lengths:

Length of Extension,cm	Number of Extensions
5	2
10	2
30	1
50	1
100	1

Connectors: For coupling the penetration piston and proving ring assembly either directly or through extension pieces.

Dial Gauge: Reading to 0.01 mm having a travel of 25 mm, for measuring the penetration of the piston.



Dial Gauge Support: Rigid and of steel, angle welded construction or light alloy pipe construction about 2 m long, of overall height 30 cm and 45 cm wide at the feet with universal or ordinary dial gauge holder adjustable anywhere along the length of the support.

Surcharge Weight: One annular metal weight of mass 5 kg and of 250 mm diameter with a central hole 53 mm in diameter. Two circular slotted weights of mass 5 kg and of diameter 215 to 250 mm with a central hole and slot width of 53 mm. Two circular slotted weights of mass 10 kg and of diameter 215 to 250 mm with a central hole and slot width of 53 mm.

Miscellaneous Apparatus: Other general apparatus, such as spirit level, pick, spade, scoop and brush, apparatus for moisture determination [IS 2720 (Part 2,1973)] and density determination [IS 2720, (Part 29,1975]

Procedure

The general surface area to be tested should be exposed, cleaned well to make it free of all loose and dried material and levelled. Extreme care shall be taken not to disturb the test surface. The spacing of the tests should be such that operations in one area do not disturb the soil in the other area. For testing operations this spacing may be 50 cm for the penetration piston used in the test.

If actual service conditions in the field warrant, the surface to be tested may be soaked to the desired degree (Ref. table 5.1). During the process of soaking the required surcharge weights should be kept in place. The test surface should be drained of all free water, levelled and allowed to stand for at least 15 minutes before starting further operations.

The equipment used to provide load reaction (truck, tractor, truss etc), should be so located that the centre of the beam against which the loading jack will work is over the centre of the surface to be tested. If loaded truck or tractor is used for providing the necessary reaction, the rear wheels of the truck or tractor should be completely raised by means of the track type jacks placed below the frame of the body near the wheels in order to avoid the loss of loading effort which would otherwise be spent on the flexing of the axial springs of the vehicle at the time of testing. In order to avoid accidents due to the failure of jacks near the wheels and the lifting of the vehicle at higher loads, the rear side of the body of the vehicle should be placed over two rigid supports. The screw jack with swivel should be installed to the underside of the equipment -providing reaction, at the correct position for the test. The proving ring should be connected to the bottom end of the jack and the piston connector to the bottom of the proving ring. The piston should then, be connected using, if necessary, lengths of extension pipes or rods. It should be ensured that the entire assembly is plumb and the loading jack should be clamped in position.

The surcharge annular weight of mass 5 kg should be kept in position on the surface to be tested so that when the piston is lowered, it will pass through the hole in the annular weight. The penetration piston should be seated with the smallest possible load not exceeding a total load of 40 N (or unit load of 0.02 MPa) so that full contact is established between the piston and the surface to be tested. For materials with irregular surface the piston may be seated on a thinnest practical layer of fine limestone screening or plaster of Paris spread over the surface.

While the seating load is on the piston, a 3 to 6 mm layer of clean sand should be spread over the surface to be covered by the surcharge annular weight. This helps in distributing the surcharge load over the surface uniformly.

Surcharge weights, sufficient to produce an intensity of loading, equal to the weight of the base material and pavement, except that the minimum weight applied should be 150 N including that of the annular weight [this weight gives an intensity of loading approximately equal to that in the laboratory bearing ratio test [IS 2720-16 (1987): Methods of test for soils, Part 16:Laboratory determination of CBR [CED 43: Soil and Foundation Engineering] should be applied. The penetration indicating dial should be suitably fixed for reading the penetration and the dial set to zero. A diagrammatic set up of the test is shown in Fig. 6.1.

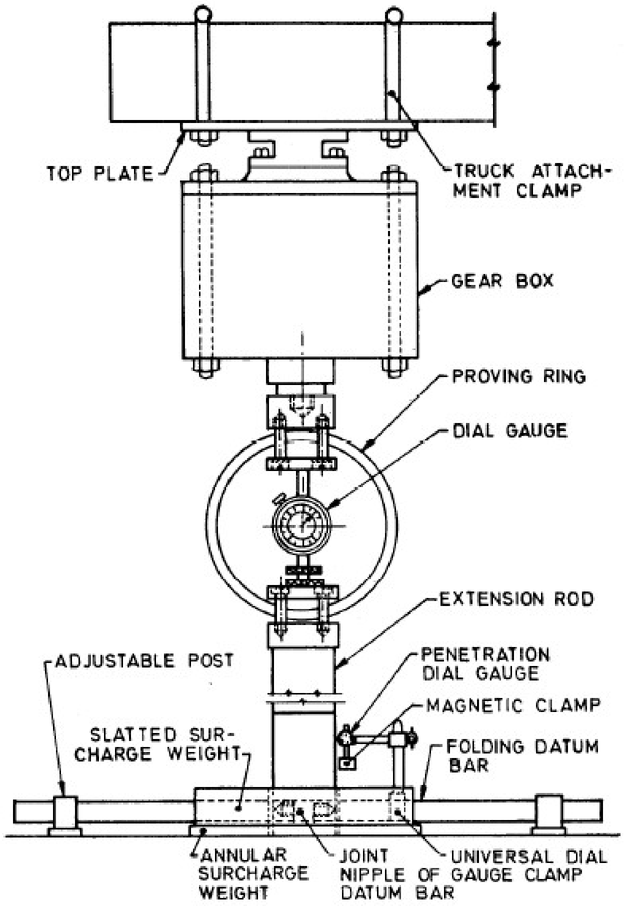


FIG. 6.1 FIELD CBR APPARATUS

Load has to be applied on the penetration piston so that the penetration is approximately 1.25 mm/min. The load readings has to be recorded at penetration of 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0, 7.5, 10.0 and 12.5 mm. The maximum load and penetration has to be recorded if it occurs for a penetration less than 12.5 mm. The set up may then be dismantled.

After the completion of the test, a sample has to be collected from the point of penetration, for moisture content determination. The moisture content has to be determined in accordance with IS 2720 (Part 2) : 4973. Besides the moisture content, the in-place density has to be determined in accordance with IS 2720 (Part 28) : 1974 or IS 2720 (Part 29) : 1975 about 15 cm away from the point of penetration.

CALCULATIONS

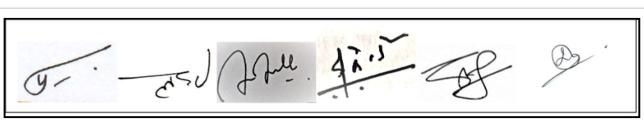
Load Penetration Curve : The load penetration curve has to be plotted (see Fig. 5.3 above). This curve may be convex upwards although the initial portion of the curve may be concave upwards due to surface irregularities. A correction has to be then applied by drawing a tangent to the curve at the point of maximum slope. The corrected curve has to be taken to be this tangent, together with the convex portion of the original curve, with the origin of strains shifted to the point where the tangent cuts the horizontal axis for penetration, illustrated in Fig. 5.3.

Bearing Ratio

Corresponding to the penetration value at which the bearing ratio is desired, corrected load values has to be taken from the load penetration curve and the bearing ratio calculated as

$$CBR = \frac{P_T}{P_S} \times 100\% \qquad \textbf{Equation 6.1}$$

where



P_T=corrected unit (or total) test load corresponding to the chosen penetration from the load penetration curve, and

P_S=unit (or total) standard load for the same depth of penetration as for P_T taken from the table 5.3.

The bearing ratios are usually calculated for penetration of 2.5 mm and 5 mm.

Report

The bearing ratio has to be reported correct to the first decimal place. The details in there commended proforma for the record of test results are given below:

PROFORMA FOR IN-PLACE BEARING RATIO TEST

Location.............................. Tested by............................
Material at the test point......................... Date..................................
Depth of tests point...

Condition of test $\frac{\text{Soaked}}{\text{Unsoaked}}$

Period of soaking, if any...
Surcharge weight used during soaking..
Moisture content...
Density...
Method used for determination of density..

Penetration Test

Surcharge weight used...

Penetration mm	Proving Ring Dial Gauge Readings	Load N	Corrected Load N
0.5			
1			
1.5			
2			
2.5			
3			
4			
5			
7			
10			
12.5			

Bearing Ratio at 2.5 mm Penetration 1 2 3 } Average   Bearing Ration of 5 mm Penetration 1 2 3 } Average

Reasons if tests is rejected:

Results of repeat test, if conducted………………………………………………….

Number of field tests

Three in-place bearing ratio tests have to be performed at each location to be tested. However, if the results of the three tests in any group do not show reasonable agreement, three additional tests have to be performed at the same location and numerical average of the six tests shall be used as the bearing ratio at that location. A reasonable agreement between the minimum and maximum values of the three tests where the bearing ratio is less than 10% permits a tolerance of 3%, from 10% to 30% a tolerance of 5%, from 30% to 60% a tolerance of 10%, and greater than 60%, a tolerance of 25%. If it is confirmed that a single value is erratic for any reason, that value should be discarded and another test shall be performed.

6.2 Dynamic Cone Penetrometers

Cone penetrometer tests should be restricted to fine- grained subgrades to avoid misleading results as a result of the influence of large particles. The dynamic cone penetrometer (DCP) test was developed by Transport and Road Research Laboratory (TRRL), England. The DCP is an instrument designed for the rapid in-situ measurement of the structural properties of existing road pavements constructed with unbound materials. It is also used for determining the in-situ CBR value of compacted soil sub-grade beneath the existing road pavement. Continuous measurements can be made down to a depth of 800 mm or, when an extension rod is fitted, to a depth of 1200 mm. Where pavement layers have different strengths the boundaries can be identified and the thickness of the layers determined.

Correlations have been established between measurements with DCP and California Bearing Ratio (CBR) so that results can be interpreted and compared with CBR specifications for pavement design. Agreement is generally good over most of the range but differences are apparent at low values of CBR, especially for fine-grained materials. A typical test takes only a few minutes and therefore the instrument provides a very efficient method of obtaining information which would normally require the digging of test pits.

Apparatus

The DCP uses an 8kg weight dropping through a height of 575 mm and a 60" cone

The apparatus is assembled as shown in Figure 6.2. It has the following parts :

a) Handle

b) Top Rod

c) Hammer (8kg)

d) Anvil

e) Handguard Cursor

f) Bottom Rod

g) 1 Meter rule

h) 60˚ Cone

i) Spanners and tommy bar are used to ensure that the screwed joints are dept tig

The following joints should be secured with loctite or similar non-hardening thread locking co

(i) Handle/top rod

(ii) Anvil/bottom rod

(iii) Bottom rod/cone

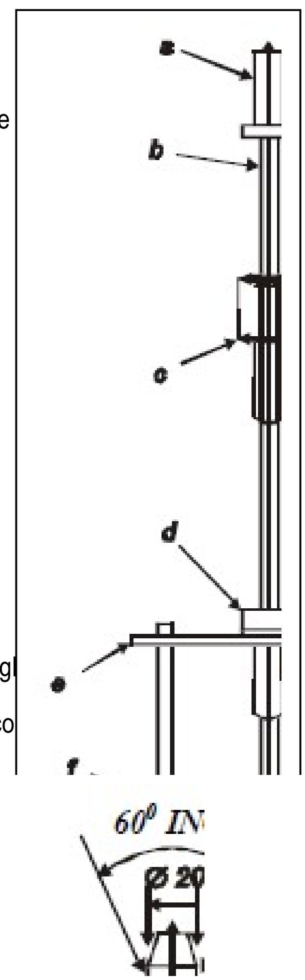


Figure 6.2: DCP Apparatus

Procedure

a) After assembly, the zero reading of the apparatus is recorded. This is done by stan such as concrete, checking that it is vertical and then entering the zero reading in the appropriate place on the proforma (Form A).

b) The instrument is held vertical and the weight carefully raised to the handle. Care should be taken to ensure that the weight is touching the handle, but not lifting the instrument, before it is allowed to drop and that the operator lets it fall freely and does not lower it with his hands.

If during the test the DCP leaves the vertical no attempt should be made to correct this as contact between the bottom rod and the sides of the hole will give rise to erroneous results.

c) It is recommended that a reading should be taken at increments of penetration of about 10mm. However, it is usually easier to take a scale reading after a set number of blows. It is therefore necessary to change the number of blows between readings according to the strength of the layer being penetrated

There is no disadvantage in taking too many readings, however if readings are taken too infrequently, weak spots may be missed and it will be more difficult to identify layer boundaries accurately, hence important information will be lost.

d) After completing the test the DCP is removed by gently tapping the weight upwards against the handle. Care should be taken when doing this as if it is done too vigorously the life of the instrument will be reduced.

Penetration rates as low as 0.5 mm/blow are acceptable but if there is no measurable penetration after 20 consecutive blows it can be assumed that the DCP will not penetrate the material. Under these circumstances a hole can be drilled through the layer using an electric or pneumatic drill or by coring. The lower layer can then be tested in the normal way. If only occasional difficulties are experienced in penetrating granular materials it is worthwhile repeating any failed tests a short distance away from the original test point.

Cone should be replaced when its diameter is reduced by 10 percent.

Calculation and expression of results.

The results of the DCP test are usually recorded on a field data sheet similar to that shown in Form A. The results can then either be plotted by hand or processed by computer. The boundaries between layers are easily identified by the change in the rate of penetration. The thickness of the layers can usually be obtained to within 10 mm except where it is necessary to core (or drill holes) through strong materials to obtain access to the lower layers. In these circumstances the top few millimeters of the underlying layer is often disturbed slightly and appears weaker than normal.

ASTM-D6951-09 relation: $\log_{10} CBR = 2.465 - 1.12 \log_{10} N$ **Equation 6.2**

Where, N = mm/blow

Form A

DCP - TEST

SITE/ROAD : *KENANA, SUDAN*

TEST NO. *6*

SECTION NO/CHAINAGE : *2/48* ZERO READ

DIRECTION : *SOUTH* STARTED TI

TOP OF BIT

WHEEL PATH : *VERGE SIDE*

No. Blows	S Blows	mm	No. Blows	S Blows	mm	N
0	*0*	*63*	*5*	*160*	*408*	

6.3 CBR Determination for Low Volume Roads (Alternative Methods)

Method A :Based on Soil Classification

Table 6.1 : Typical Soaked CBR

Sub-grade Soil	IS Soil Classification	Typical Soaked CBR %
Highly Plastic Clay and Silts	CH, MH	* 2-3
Silty Clay and Sandy Clay	ML, MI-CL, CI	4-5
Clayey Sands and Silty Sands	SC, SM	6-10

* For Expansive Clay, CBR = 2%

Method B : Quick Estimation

Plastic Soil

CBR = 75 / [1 + 0.728 x WPI]

Where, WPI = Weighted Plastic Index = $P_{0.075}$ x PI

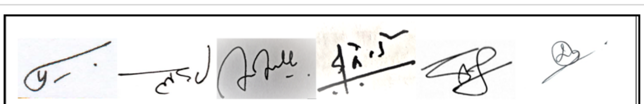
$P_{0.075}$ = % passing 0.075mm Sieve in decimal

PI = Plasticity Index of Soil in %

Non-Plastic Soil

CBR = 28.091 x $(D_{60})^{0.3581}$

Where, D_{60} = Diameter in mm of Grain Size corresponding to 60 % finer



7. Selection of Subgrade CBR for Pavement Design

The CBR values of the subgrade soil varies along a road alignment even on a homogenous section. 90th percentile CBR is recommended by IRC 37-2018 for National Highways and State Highways. For other categories of roads, the design can be done based on the 80th percentile CBR value if the design traffic is less than 20 msa and based on 90th percentile CBR if the design traffic is 20 msa or more.

Asphalt Institute of USA has recommended 87.5 percentile subgrade modulus for design traffic greater than one msa. If the data is very large, the CBR values can be grouped for homogenous sections.

Example 7.1 90th percentile CBR determination

Field Data

Chainage	Field CBR
0	3.5
200	6
400	4.3
600	4.9
700	5.5
800	8.9
1000	7
1200	7.3
1400	3
1600	4.6
1800	5.7
2000	5.9

Processed Data

CBR Value in ascending order	% ≥ to each value
3	100
3.5	91.7
4.3	83.3
4.6	75.0
4.9	66.7
5.5	58.3
5.7	50.0
5.9	41.7
6	33.3
7	25.0
7.3	16.7
8.9	8.3

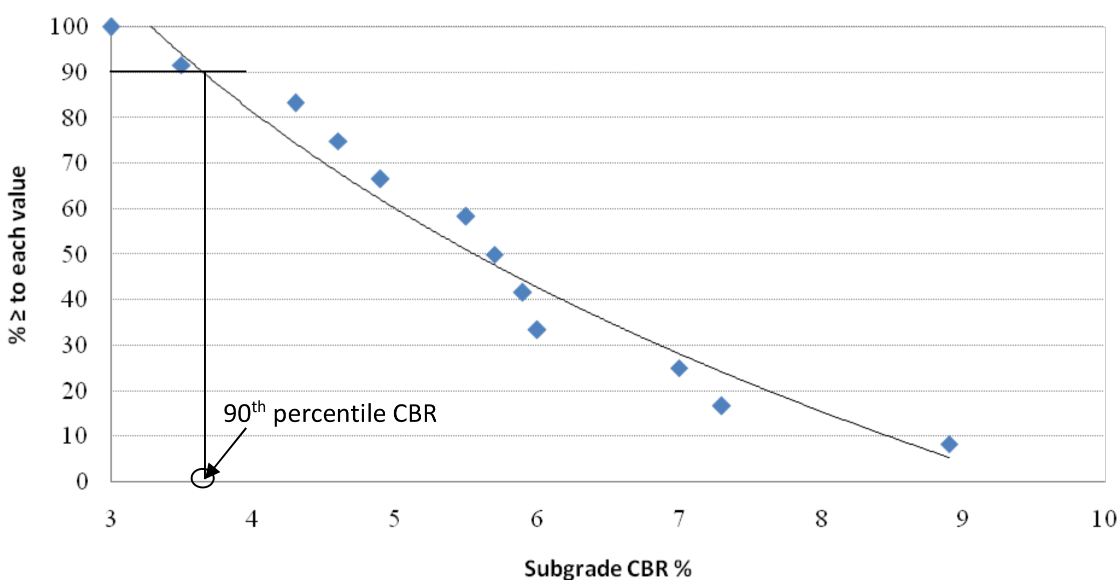
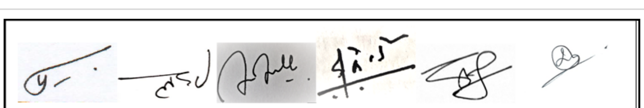


Figure : 7.1 Evaluation of Subgrade CBR for Pavement Design



Where there is significant difference between the CBRs of the selected subgrade and embankment soils, the design should be based on effective CBR.

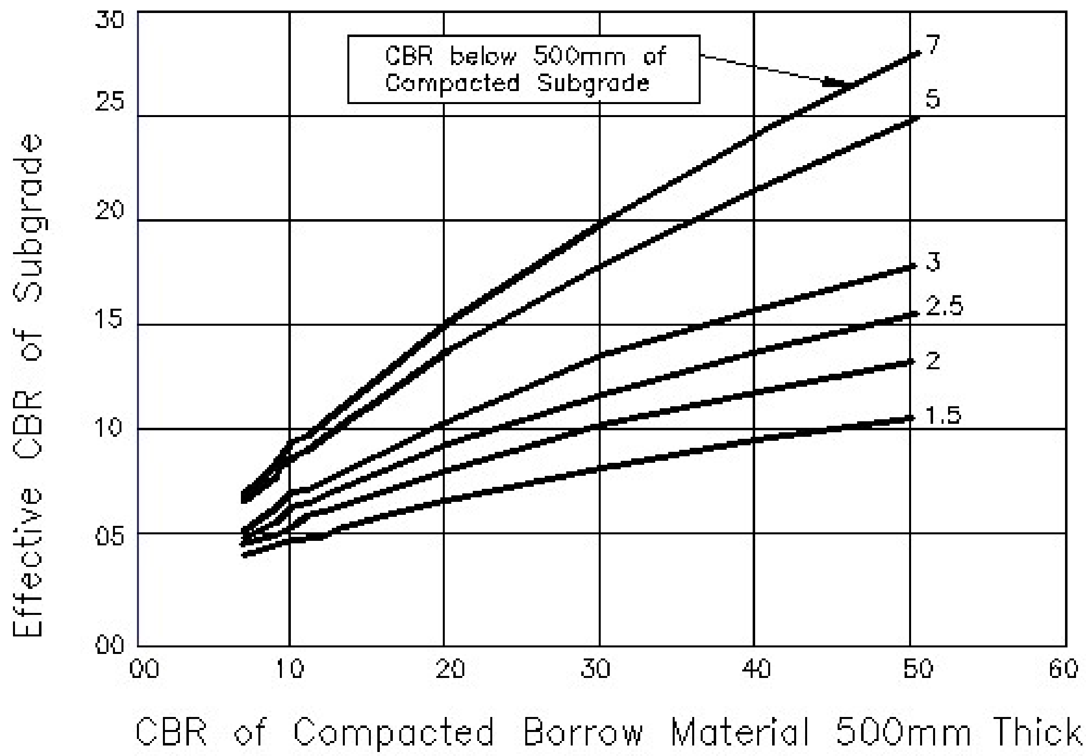


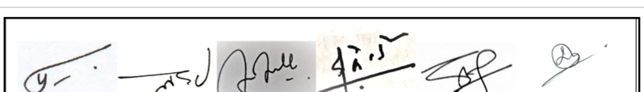
Figure 7.2: Effective Borrow Material 500mm Thick

8. Adoption of Presumptive CBR Values

This approach may be used when no other relevant information is available. It is particularly useful for lightly-trafficked roads where extensive investigations are not warranted, and also when conducting preliminary designs for all roads. Typical presumptive values of CBR are given in Table 8.1. However, such values should only be utilised on the basis that the information will be supplemented by taking account of local experience.

Table 8.1: Typical presumptive subgrade design CBR values(Ref. IRC 36:2010)

General Soil Type	USC Soil Type	Value as Subgrade when not subjected to frost action	Unit Dry Weight,g/cm3	CBR Range
Coarse-grained soils	GW	Excellent	2.0 – 2.24	40 – 80
	GP	Good to Excellent	1.76 – 2.24	30 – 60
	GM	Good	1.84 – 2.16	20 – 60
	GC	Good	2.08 – 2.32	20 – 40
	SW	Good	1.76 – 2.08	20 – 40
	SP	Fair to Good	1.68 – 2.16	10 – 40
	SM	Fair	1.60 – 2.08	10 – 40
	SC	Poor to Fair	1.60 – 2.16	5 – 20
Fine-grained soils	ML	Poor to Fair	1.44 – 2.08	15 or less
	CL LL < 50%	Poor to Fair	1.44 – 2.08	15 or less
	OL	Poor	1.44 – 1.68	5 or less
	MH	Poor	1.28 – 1.68	10 or less
	CH LL > 50%	Poor to Fair	1.44 – 1.84	15 or less
	OH	Poor to very poor	1.28 – 1.76	5 or less



9. Frost Consideration

Frost action can be quite detrimental to pavements and refers to two separate but related processes:

- Frost heave. An upward movement of the subgrade resulting from the expansion of accumulated soil moisture as it freezes.
- Thaw weakening. A weakened subgrade condition resulting from soil saturation as ice within the soil melts.

Figure 9.1. Frost heave on a city street in central Sweden.

Frost heaving of soil is caused by crystallization of ice within the larger soil voids and usually a subsequent extension to form continuous ice lenses, layers, veins, or other ice masses. An ice lens grows through capillary rise and thickens in the direction of heat transfer until the water supply is depleted or until freezing conditions at the freezing interface no longer support further crystallization. As the ice lens grows, the overlying soil and pavement will “heave” up potentially resulting in a cracked, rough pavement (see Figure 9.1). This problem occurs primarily in soils containing fine particles (often termed “frost susceptible” soils), while clean sands and gravels (small amounts of fine particles) are non-frost susceptible (NFS). Thus, the degree of frost susceptibility is mainly a function of the percentage of fine particles within the soil. Many agencies classify materials as being frost susceptible if 10 percent or more passed a 0.075 mm (No. 200) sieve or 3 percent or more passed a 0.02 mm (No. 635) sieve. Figure 9.2 illustrates the formation of ice lenses in a frost susceptible soil.

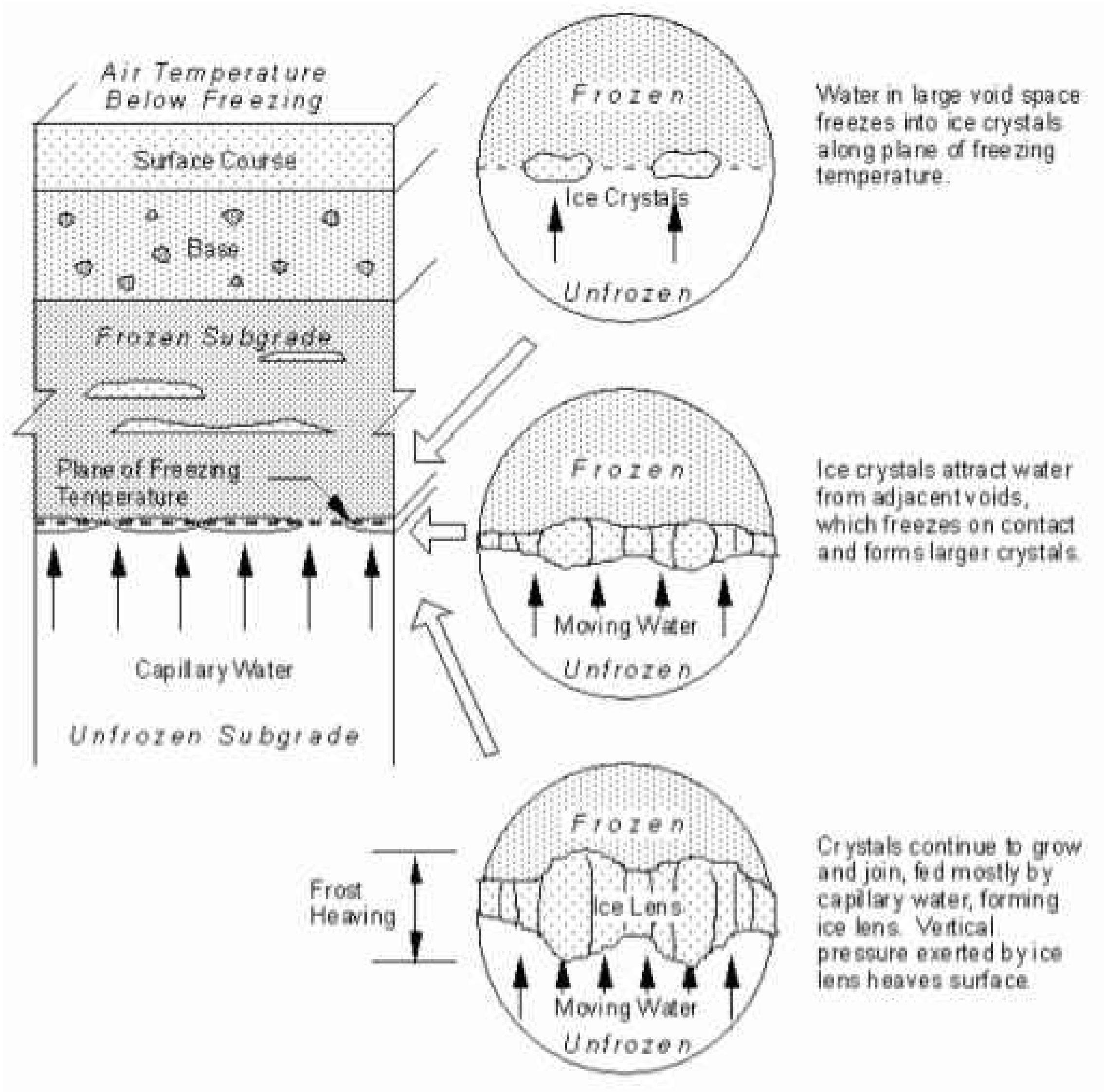


Figure 9.2. Formation of ice lenses in a pavement structure.

The three elements necessary for ice lenses and thus frost heave are:

1. Frost susceptible soil (significant amount of fines).
2. Subfreezing temperatures (freezing temperatures must penetrate the soil and, in general, the thickness of an ice lens will be thicker with *slower* rates of freezing).
3. Water (must be available from the groundwater table, infiltration, an aquifer, or held within the voids of fine-grained soil).

Remove any of the three conditions above and frost effects will be eliminated or at least minimized. If the three conditions occur uniformly, heaving will be uniform; otherwise, differential heaving will occur resulting in pavement cracking and roughness. Differential heave is more likely to occur at locations such as:

- Where subgrades change from clean not frost susceptible (NFS) sands to silty frost susceptible materials.
- Abrupt transitions from cut to fill with groundwater close to the surface.
- Where excavation exposes water-bearing strata.

- Drains, culverts, etc., frequently result in abrupt differential heaving due to different backfill material or compaction and the fact that open buried pipes change the thermal conditions (i.e., remove heat resulting in more frozen soil).

Additional factors which will affect the degree of frost susceptibility (or ability of a soil to heave):

- Rate of heat removal.
- Temperature gradient
- Mobility of water (e.g., permeability of soil)
- Depth of water table
- Soil type and condition (e.g., density, texture, structure, etc.)

The Casagrande Criterion

In 1932, Dr. Arthur Casagrande proposed the following widely known rule-of-thumb criterion for identifying potentially frost susceptible soils:

"Under natural freezing conditions and with sufficient water supply one should expect considerable ice segregation in non-uniform soils containing more than 3% of grains smaller than 0.02 mm, and in very uniform soils containing more than 10 percent smaller than 0.02 mm. No ice segregation was observed in soils containing less than 1 percent of grains smaller than 0.02 mm, even if the groundwater level is as high as the frost line."

Application of the Casagrande criterion requires a hydrometer test of a soil suspension (in water) to determine the distribution of particles passing the 0.075 mm sieve and to compute the percentage of particles finer than 0.02 mm.

Thaw Weakening

Thawing is essentially the melting of ice contained within the subgrade. As the ice melts and turns to liquid it cannot drain out of the soil fast enough and thus the subgrade becomes substantially weaker (less stiff) and tends to lose bearing capacity. Therefore, loading that would not normally damage a given pavement may be quite detrimental during thaw periods (e.g., spring thaw). Figure 9.3 is an example of typical pavement deflection changes throughout the year caused by winter freezing and spring thawing. Figure 9.4 shows pavement damage as a result of thaw weakening.

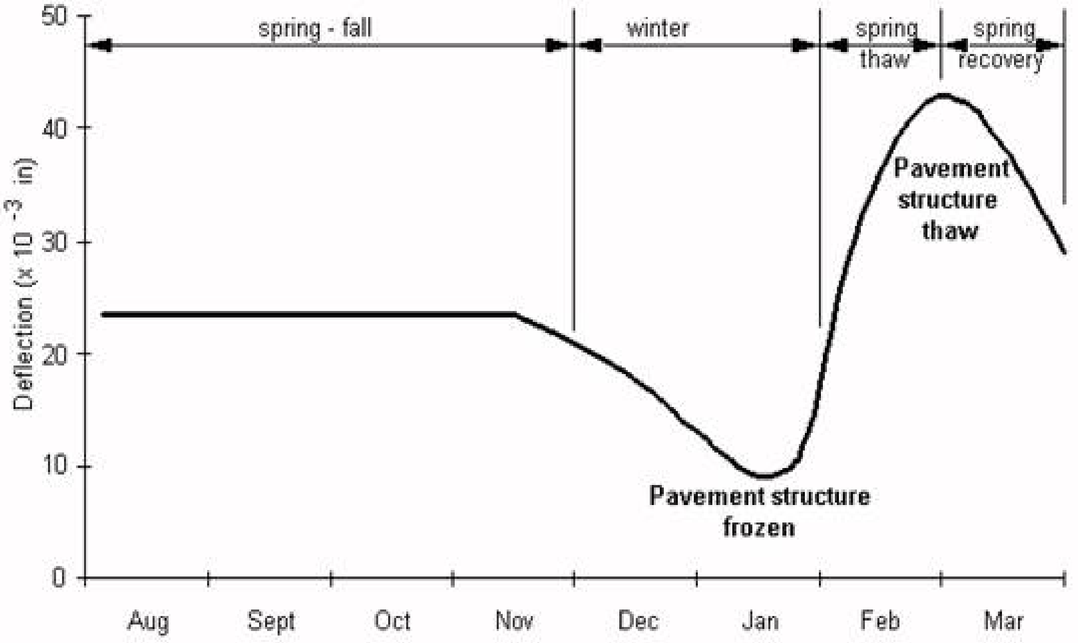


Figure 9.3. Example of Typical pavement deflections illustrating seasonal pavement strength changes (on a portion of State Route 172 in Washington State).

Figure 9.4. Freeze-thaw damage.

Thawing can proceed from the top downward, or from the bottom upward, or both. How this occurs depends mainly on the pavement surface temperature. During a sudden spring thaw, melting will proceed almost entirely from the surface downward. This type of thawing leads to extremely poor drainage conditions. The frozen soil beneath the thawed layer can trap the water released by the melting ice lenses so that lateral and surface drainage are the only paths the water can take.

Tabor (1930[1]) also noted an added effect:

"The effects of refreezing after a thaw are also accentuated by the fact that the first freeze leaves the soil in a more or less loosened or expanded condition."

This observation shows that (1) the reduced density of base or subgrade materials helps to explain the long recovery period for material stiffness or strength following thawing, and (2) refreezing following an initial thaw can create the potential for greater weakening when the "final" thaw does occur.

<u>Sources of Water</u>

The two basic forms of frost action (frost heave and thawing) both require water. Water sources can be separated into two broad categories:

1. **Surface water**. Enters the pavement primarily by infiltration through surface cracks and joints, and through adjacent unpaved surfaces, during periods of rain and melting snow and ice. Many crack-free pavements are not entirely impermeable to moisture.
2. **Subsurface water**. Can come from three primary sources:
    - Groundwater table (or perched water table).
    - Moisture held in soil voids or drawn upward from a water table by capillary forces.
    - Moisture that moves laterally beneath a pavement from an external source (e.g., pervious water bearing strata, etc.).

<u>Estimation of Freezing or Thawing Depths in Pavements</u>

This section discusses freeze depth estimation techniques. Such an estimate is helpful in designing for frost conditions, but oversimplifies the complex conditions that accompany various pavement materials, depths of freeze, and water sources. Basic terminology is contained on a separate page. All units will be in U.S. customary due to the source material. Two formulas are presented on linked pages:

- **Modified Berggren Formula**
- **Stefan Formula**

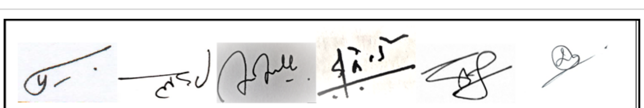
Modified Berggren Formula

The modified Berggren formula was developed in the early 1950s to address the shortcomings of the Stefan formula. The modified Berggren formula assumes that the soil is a semi-infinite mass with uniform properties and existing initially at a uniform temperature (Ti). It is further assumed that surface temperature is suddenly changed from Ti to Ts (below freezing). The modified Berggren formula is simply the Stefan formula corrected for the effects of temperature changes in the soil mass:

$$x = \lambda \sqrt{\frac{48k_{a}}{}}$$

Equation . 9.1

where:	x	equals	depth of freeze or thaw, (ft))
	λ	equals	dimensionless coefficient which takes into consideration the effect of temperature changes in the soil mass (i.e., a fudge factor). Corrects the Stefan formula for the



neglected effects of volumetric heats (accounts for "sensible heat" changes)

k_{avg} equals thermal conductivity of soil, average of frozen and unfrozen (BTU/hr • ft • °F)

n equals conversion factor for air freezing (or thawing) index to surface freezing (or thawing) index

FI equals air freezing index (°F • days)

TI equals air thawing index (°F • days)

L equals latent heat (BTU/ft^3) depends only on amount of water in a unit volumne of soil.

Determination of λ

l can be determined by chart (Figure 9.5) based on inputs of a (thermal ratio) and m (fusion parameter).

λ equals f (FI (or TI), mean annual air or ground temperature, thermal properties of soil)

equals f(α,μ) and can be read from Figure 9.5

μ equals fusion parameter

$$= \left(|T_f - T_s|\right.$$

C equals average volumetric heat capacity of a soil (BTU/ft^3 • °F)

L equals latent heat (BTU/ft^3) = 1.434 x water content x soil dry density in pcf

$|T_f - T_s|$ equals surface freezing (or thawing) index, nFI (or nTI) divided by length of freezing (or thawing) season. Represents temperature differential between average surface temperature and 32 °F taken over the entire freeze (or thaw) season.

equals $\frac{nFI}{d}$ or

d equals length of freezing or thawing duration. For example, if the winter freezing season is December through February, then the duration of freezing (d) equals about 90 days.

T_f equals 32 °F

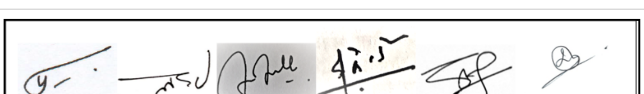
T_s equals average surface temperature for the freezing (or thawing) period

α equals thermal ratio

$$= \left|\frac{\overline{T} - }{T}\right.$$

T equals average annual air or ground temperature

$|T - T_f|$ equals represents the amount that the mean annual temperature exceeds (or is less than) the freezing point of the soil moisture (assumed to be 32 °F).



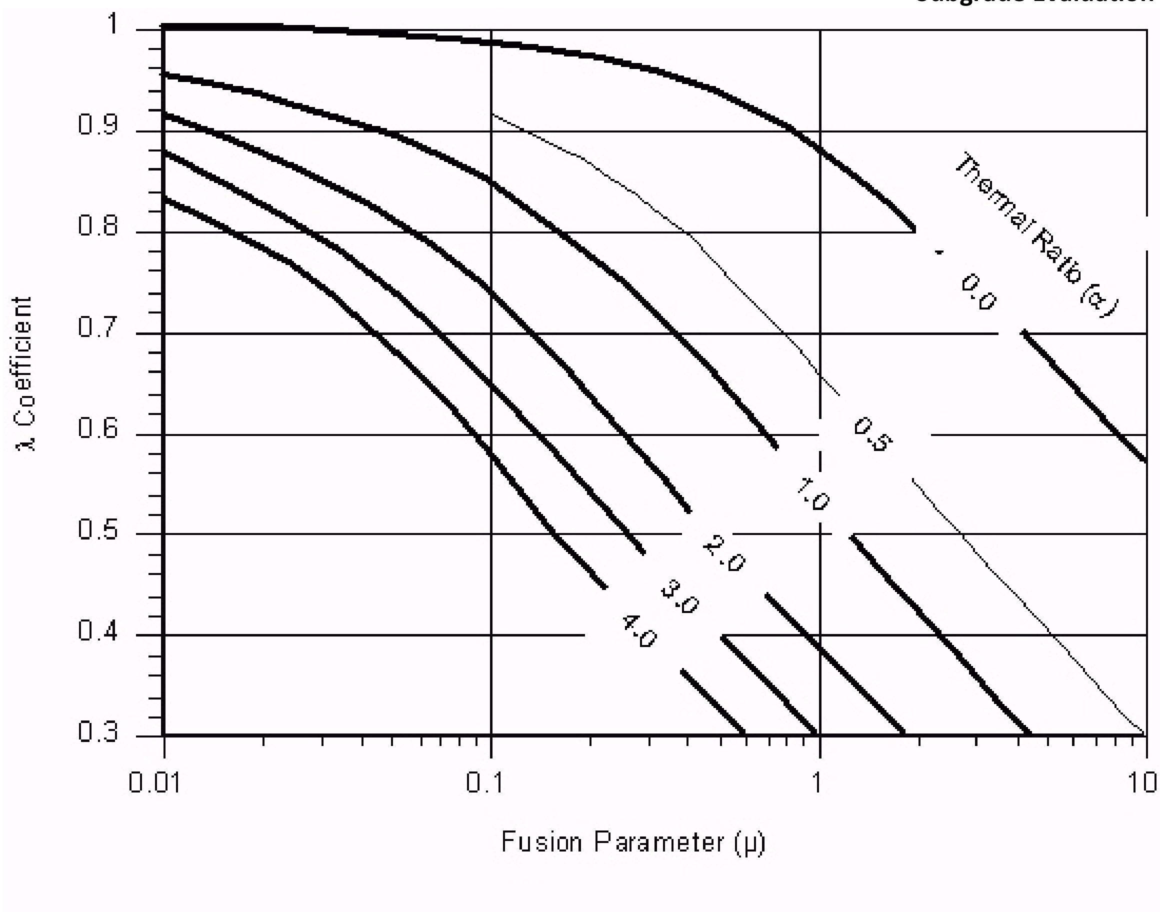


Figure 9.5: I coefficient in the Modified Berggren Formula.

Modified Berggren Formula Example

Determine the depth of frost penetration into a homogeneous sandy silt for the following conditions:

Given:

- Mean annual temperature = 48 °F
- Surface freezing index = nFI = 750 °F days
- Duration of freezing season = d = 100 days

Soil properties:

- Dry density = ρ_d = 100 lb/ft^3
- Water content = w_c = 15% “

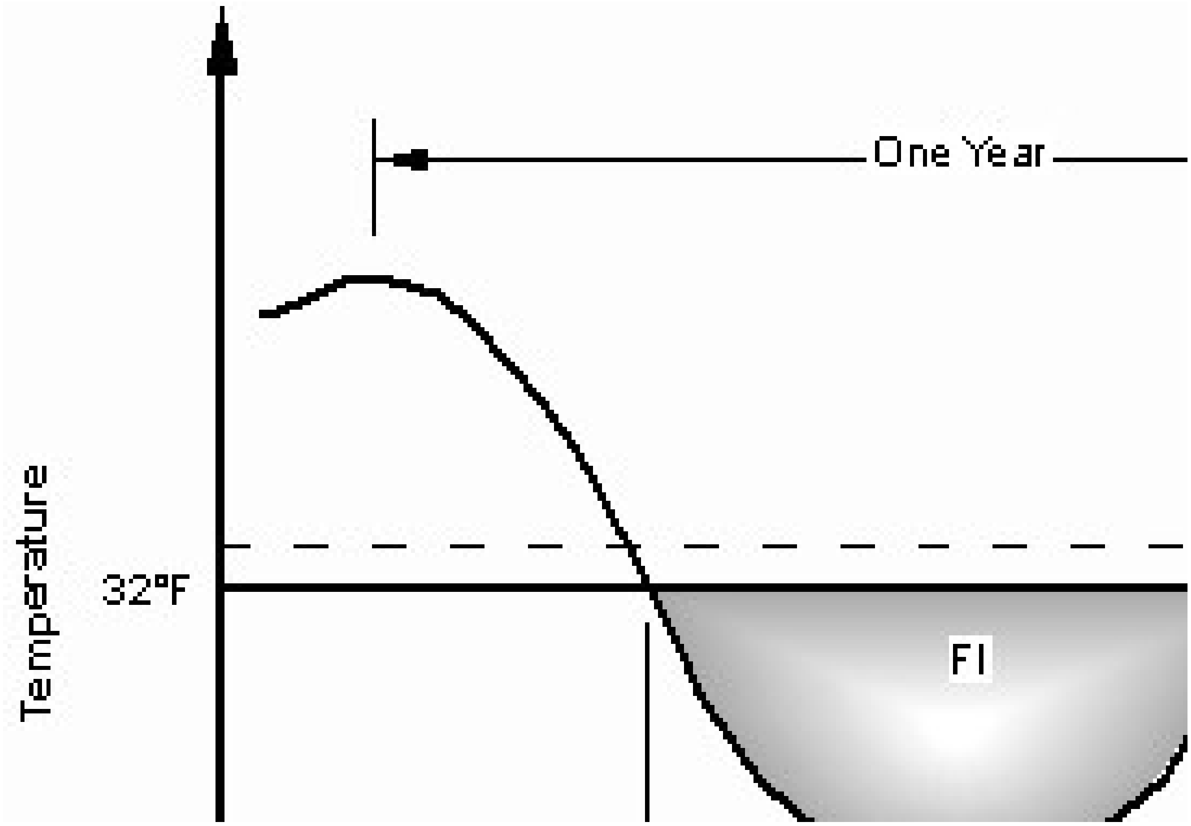


Figure 9.6: Pictorial representation of variables involved with I.

Solution

- Calculate soil thermal properties

Volumetric latent heat of fusion:
L = (144 BTU/lb) (100 lb/ft^3) (15/100) = 2160 BTU/ft^3

Average volumetric specific heat:
C_{avg} = 100 (lb/ft^3) (0.15) = 28.2 BTU/ft^3 • °F

Average thermal conductivity:
k_f @ 0.80 BTU/hr • ft • °F
k_u @ 0.72 BTU/hr • ft • °F

Therefore, k_{avg} = 0.76 BTU/hr • ft • °F

- Calculate λ (recall λ = f(α, μ))

$$\alpha = \left|\frac{\overline{T}-T_f}{\ldots}\right| = \frac{\left|\overline{T}-T_f\right|(d)}{\ldots} = \frac{|48-\ldots}{\ldots}$$

$$\mu = \left(\left|T_f - T_s\right|\right)\left(\frac{C_{avg}}{\ldots}\right) = \frac{nFI}{\ldots}\left(\frac{C_{avg}}{\ldots}\right) = \left(\ldots\right.$$

From the chart, λ= 0.74

- Calculate depth of freezing

$$x = \lambda_{\cdot}\sqrt{\frac{48k_{avg}\ nFI}{}} = (0.74)\sqrt{\frac{48\,(0.76)\,(7}{}}$$

Stefan Formula

Some of the first studies of freeze/thaw depth were made by Josef Stefan in 1889, in connection with ice formation and melting in the Polar oceans (Paynter, not date given). In this formula it is assumed that the latent heat of soil moisture is the only heat that must be removed when freezing the soil. Thus, thermal energy stored as volumetric heat and released as soil-temperatures drop to and below freezing is not considered. Because volumetric heat is neglected, the Stefan Formula tends to overestimate frost depth in temperate zones (Paynter, no date given). The latent heat supplied by the soil moisture as it freezes a depth dx in time dt = rate at which heat is conducted to the ground surface. This is illustrated in the sketch below:

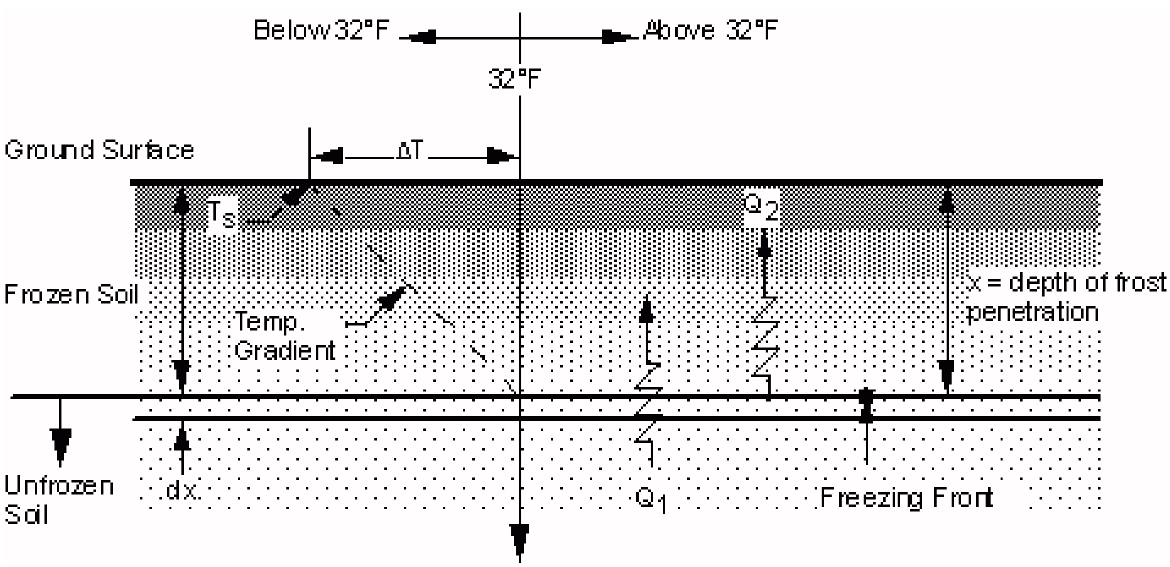


Figure 9.7: Stefan Formula Diagram

Heat removal process can be represented by

$$Q_1 = L$$ **Equation . 9.2**

(heat released by freezing a layer of soil dx thick in time dt)

$$Q_2 = \frac{\Delta T}{R} = \frac{}{}$$ **Equation . 9.3**

(heat conducted through frozen layer)

and Q1 = Q2 so $L\frac{dx}{\cdot} = k_f$ **Equation . 9.4** by integrating and solving for x

, $$x = \sqrt{\frac{2k_f}{-} \int t} \qquad \textbf{Equation . 9.5}$$

$\int \Lambda 7$is in units of °F • hr and is called surface freezing index. The freezing index is normally expressed as °F • days. Thus, rewrite the equation and add an "n" factor which results in the Stefan formula:

$$x = \sqrt{\frac{48k_f}{-}} \qquad \textbf{Equation . 9.6}$$

Mitigating Frost Action

Mitigating of frost action and its detrimental effects generally involves structural design considerations as well as other techniques applied to the base and subgrade to limit the effects of frost action. The basic methods used can be broadly categorized into the following techniques:

- **Limit the depth of frost into the subgrade soils**. This is typically accomplished by specifying the depth of pavement to be some minimum percentage of the frost depth. By extending the pavement section well into the frost depth, the depth of frost-susceptible subgrade under the pavement (between the bottom of the pavement structure and frost depth) is reduced. The assumption is that a reduced depth of soil under frost action will cause correspondingly less damage.
- **Removing and replacing frost-susceptible subgrade.** Ideally the subgrade will be removed at least down to the typical frost depth. Removing frost-susceptible soils removes frost action.
- **Design the pavement structure based on reduced subgrade support**. This method simply increases the pavement thickness to account for the damage and loss of support caused by frost action.
- **Providing a capillary break**. By breaking the capillary flow path, frost action will be less severe because as Tabor (1930[1]) noted, frost heaving requires substantially more water than is naturally available in the soil pores.
- **IRC 37 suggest as general rule , increase the depth of construction correspond to the depth of frost penetration, at least 45 cm** .

Freezing and Thawing Implications for Maintenance Operations

The calculated freezing index (FI) and thawing index (TI) can be used to estimate the depth of freeze at a specific site and the resulting thaw. Maintenance personnel can use the TI to assess the need for seasonal load limits (Figure 5). The following general guidelines relative to spring highway load restrictions were developed and evaluated by a study in Washington State (Rutherford et al., 1985[2]; Mahoney et al., 1986[3]):

- **Where to apply load restrictions**. If pavement surface deflections are available to an agency, spring thaw deflections greater than 45 to 50 percent of summer deflections suggest a need for load restriction. Further, considerations such as depth of freezing (generally areas with air Freezing Indices of 400 °F-days or more), pavement surface thickness, moisture condition, type of subgrade, and local experience should be considered. Subgrades with Unified Soil Classifications of ML, MH, CL, and CH will result in the largest pavement weakening.

- **Amount of load reduction**. The minimum load reduction level should be 20 percent. Load reductions greater than 60 percent generally are not warranted based on potential pavement damage. A load reduction range of 40 to 50 percent should accommodate a wide range of pavement conditions.
- **When to apply load restrictions**. Load restrictions “should” be applied after accumulating a Thawing Index (TI) of about 25 °F-days (based on an air temperature datum of 29 °F) and “must” be applied at a TI of about 50 °F-days (again based on an air temperature datum of 29 °F). Corresponding TI levels are less for thin pavements (e.g., two inches of HMA and six inches of aggregate base or less) in that the “should apply” TI level is 10 °F-days and the “must” TI level is 40 °F-days.
- **When to remove load restrictions**. Two approaches are recommended, both of which are based on air temperatures. The duration of the load restriction period can be directly estimated by the following relationship, which is a function of Freezing Index (FI):

Duration (days) = 25 + 0.01 (FI)

The duration can also be estimated by use of TI and the following rough relationship:

TI = 0.3 (FI)

<u>Frost Action Summary</u>

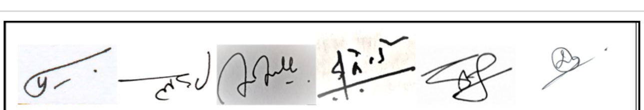
Frost action is a critical pavement structural design concern in those parts of the country that regularly experience ground freezing. Without proper precautions, severe frost action can destroy a new pavement in a matter of one or two years. In taking the proper precautions, there are two basic types of frost action with which to contend:

1. **Frost heave**. Results from accumulation of moisture in the soil during the freezing period. These accumulations (ice lenses) expand perpendicular to the direction of heat flow and push the pavement up, often causing severe cracking.
2. **Thaw weakening**. Once a subgrade is frozen it can be severely weakened when it thaws (usually in the spring time). Therefore, loading that would not normally damage a given pavement may be quite detrimental during thaw periods.

Frost action can be further characterized by the typical depth to which the subgrade freezes in a particular area. This depth can be estimated by several equations including the Stefan formula and the modified Berggren formula. Once this depth is known, it can be used as a pavement structural design input to mitigate the detrimental effects of frost action. Mitigation techniques can be classified into four broad categories:

1. Limit the depth of frost-susceptible material under the pavement structure.
2. Remove and replace the frost-susceptible subgrade.
3. Design the pavement structure based on reduced subgrade support.
4. Force a break in the groundwater’s capillary path.

If frost action cannot be adequately mitigated, severe pavement damage (in the case of frost heave) or a loss of bearing capacity (in the case of thaw weakening) can result. Maintenance options to correct these problems are limited to pavement repair or replacement (in the case of frost heave) or limiting pavement loading during spring thawing (in the case of thaw weakening).



10. Limiting Subgrade Strain Criterion

In the mechanistic-empirical design of flexible pavements , the pavement is designed to limit the vertical compressive strain at the top of the subgrade to a tolerable level throughout the life of the pavement.

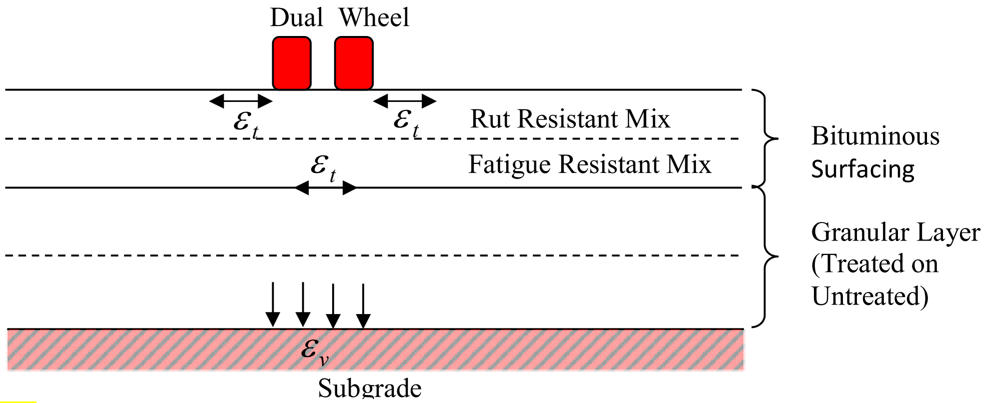


Figure: 10.1 Strain at different pavement layers

Under the vehicle wheel load, the tensile strain, ε_t, at the bottom of the bituminous layer and the vertical subgrade strain, ε_v, on the top of the subgrade are conventionally considered as critical parameters for pavement design to limit cracking and rutting in the bituminous layers and non-bituminous layers respectively.

The strain induced is mostly elastic (i.e. recoverable). However, each vertical strain induced by traffic loading is not fully recoverable, and hence, after many load applications, permanent deformation accumulates at the subgrade level, and also, though generally to a much lesser extent, throughout all the pavement layers. These permanent deformations manifest themselves as rutting in the wheel paths, although, due to the inherent variability of the subgrade and pavement materials and construction techniques, surface roughness increases as the magnitude of this deformation increases.

Note that the vertical compressive strain at the top of the subgrade is taken as a determinant for surface rutting in the unbound portions of the pavement structure. Monismith, Sousa and Lysmer (1988) express the logic for the use of the subgrade strain as a measure of surface rutting:

In pavement materials the magnitude of the plastic strain is directly proportional to the magnitude of the (vertical) elastic strain. In a pavement system the elastic strain increases from the subgrade to the surface. Accordingly, by setting the elastic strain at the subgrade surface at a specific value, the elastic strain in the components above this plane are controlled as are the values for the associated plastic strains. Integration of the plastic strains over the pavement section provides a measure of the permanent deformation (rut depth) which will occur at the pavement surface.

The limiting strain criterion for the subgrade is given in Equation 10.1.

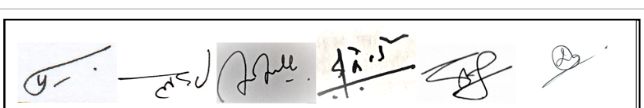
$$N = \left[\frac{9150}{\varepsilon_v}\right]^7$$

Equation 10.1

where,

N = the allowable number of repetitions of a Standard Axle at this strain before an unacceptable level of pavement surface deformation develops (units of ESAs)

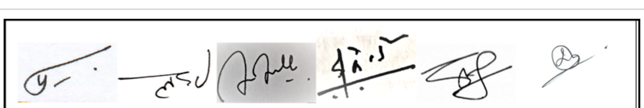
ε_v = the vertical compressive strain (in terms of microstrain), developed under a Standard Axle, at the top of the subgrade



IRC 37-2018 has suggested the limiting strain criterion for the subgrade is given in Equation 10.2.

$$\left.\begin{aligned} N &= 4.1656\times10^{-08}\left[\frac{1}{\varepsilon_v}\right]^{4.5337} \qquad at\ \ 80\%\ \ reliabilitylevel \\ N &= 1.41\times10^{-08}\left[\frac{1}{\varepsilon_v}\right]^{4.5337} \qquad at\ \ 90\%\ \ reliabilitylevel \end{aligned}\right\} \qquad \textbf{Equation 10.2}$$

If the vertical strain of the subgrade is less than 200 micro strains, there will be little rutting in subgrade.



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1. https://pavementinteractive.org
2. IRC:37-2018, Guidelines for the Design of Flexible Pavements, Indian Roads Congress, New Delhi
3. IRC:37-2012, Guidelines for the Design of Flexible Pavements, Indian Roads Congress, New Delhi
4.